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A STUDY OF SCOUR AT OBSTRUCTIONS IN NON-COHESIVE BED

P. Andru

1956

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ABSTRACT

"The main objectives of the investigation described herein were to:-

i. Obtain practical rules for estimating the scour to expect at various obstacles or constraints in a river of mobile bed.

ii. Observe the settlement of stone aprons laid to protect such obstacles or constraints from undermining.

The need for the investigation lies in the scanty information available, and the practical importance of being able to determine, in advance, the probable quantitive consequences of obstructing a river by bridges, spurs, guide banks, pipe-lines and tunnels within the reach of scour.

The natural development of a simple set of circumstances in a model river, or a flume with mobile bed, may take several weeks. Therefore, to obtain a fair return of information for a given expenditure of time, experiments were devised to give a variety of happenings and, so far as possible, experiments on different pieces of apparatus were run concurrently.

Thus the work done fell into the following four Parts:-

Part I - Plotting, to suitable coordinates, the available field data of "worst scours" from outside sources, together with laboratory data from outside sources from the writer's and others' work at the University of Alberta. The specific object was to obtain a working rule for scaling up from

model to prototype.

Part II - Attempting to produce the general pattern of behaviour that occurred at an actual bridge where, after continual buttressing of the piers by piles of stone, and after two major floods, a pier collapsed into a scour hole as the consequence of a minor flood. The specific object here was to demonstrate the practical utility of a small model for predicting, with the aid of engineering judgement, the probable major occurrences to expect in the field.

Part III - Studying the process of apron launching upstream and downstream of a weir-like obstruction built across a river. This was done in two-dimensions in a glass sided flume, and was supplemented by some other studies of apron behaviour in the river tray. The specific objective was to obtain an understanding of the mechanism of apron launching, and to obtain coefficients for scour under circumstances on which information was not very detailed.

Part IV - Studying stone apron launching, model distortion and effects of sloping ends on protection walls at three submarine crossings with guide banks. These crossings were placed in awkward positions to develop "worst scours" by disadvantageous approach conditions, and had a subsidiary objective of verifying whether they would straighten a river.

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THE UNIVERSITY OF ALBERTA

A STUDY OF SCOUR AT OBSTRUCTIONS IN NON-COHESIVE BED

A DISSERTATION

SUBMITTED TO THE SCHOOL OF GRADUATE STUDIES

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE

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by

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September 1956

P. Andrew

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P A R T I

STUDY OF SCOUR DATA

PART I

Introduction

Scour around bridge piers, groynes, spurs, and noses of guide banks can seldom be observed visually but has to be determined by "feeling" such as sounding, manually or electronically. These soundings show analagous conditions are produced by wind in snow around poles, trees, and other isolated obstructions. Noticeably bowl shaped scour results. An obstruction such as a tunnel or apron crossing between guide banks can be compared to a corridor connection between two parallel taller buildings. The wind blowing and snow being transported between the two buildings comes upon the corridor connection. One part of the flow dives and follows the face of the corridor wall, scouring the snow for a certain depth and distance away from the face of the wall. The other part flows over the roof and then curves downwards for additional scouring. The last scour can be portrayed by eddies shed from high pavements or connected stone protection placed around bridge piers.

The main scour points that we are interested in are at bridge piers, noses of guide banks, spurs and groynes, downstream of bridge piers, along guide banks and meander bends. Studies have been made with models and field measurements of maximum depths under extreme conditions have been recorded. The writer has used these data from various sources to plot ordinates of "$d_s F_b^{y_3}$ vs q" , where d_s is the observed maximum

scour depth in feet, F_b is derived from the equation $F_b = 1.9\sqrt[3]{m}$ where m is the median grain size of the bed, and q is the discharge intensity in cusecs./ft. The master plot of all available data aimed at obtaining so large a range of q that the regime relation $d \propto q^{2/3}$ could be verified with practical accuracy in spite of the differing circumstances that caused the observed depths. When the relation had been established plots of special circumstances assumed that a formula of the type $d \propto q^{2/3}/F_b^{1/3}$ was justified.

The available record of field conditions is that of C.C. Inglis (1) for Indian plains' rivers. These records of severe scour under peak flood discharge were tabulated and plotted to show the ratio of scoured depth to the "regime depth" of the river at peak flood discharge. The regime depth d_r was calculated using the original Lacey regime equations for canals, where the relative effect of sides and beds had been averaged out for canal analysis, so was not necessarily very exact for the rivers. This depth was $0.47\sqrt[3]{Q/f}$ where f is the Lacey "silt-factor".

Inglis plotted $d_s f^{1/3}$ against Q where d_s is the observed maximum scour depth below peak flood level and Q is the peak flood discharge. The points on double logarithmic paper were fitted well by a straight line of slope 1/3 as regime theory suggests. For adding data of gravel and boulder rivers where the relative importance of bed and sides is not that of the Indian rivers general regime theory (2) is applicable and gives $d_r = \sqrt[3]{F_s/F_b^2\, Q}$, where F_s is the side factor and F_b the bed factor. But F_s is not easily assessible, and for models with rigid sides cannot be assessed. So the equivalent formula $d_r = \sqrt[3]{q^2/F_b}$ was used by the author.

To plot Inglis' results on a chart of $d_s F_b^{1/3}$ against q, the relation $q^2 = F_s/F_b Q$ has been used where F_s/F_b is assumed to be about 1/5 (on recommendation by Professor Blench.) F_b in this case is designated as the bed factor corresponding to a vanishing small charge (charge is the ratio of weight of dry sediment per second to the weight of water per second), since

the charge was believed to be small. For large charges experiments will be needed to verify if a correction is needed.

Inglis' field data and some model data are reproduced in Appendix A. They are retabulated on the $\sqrt[3]{q^2/F_b}$ system in Tables 1, 2, 3. Special experiments made by him with a model of the Hardinge Bridge, with sands of different sizes, gave a relation between relative breadth of pier and relative depth of scour; data from these are retabulated in Table 4.

An article "A Generalized Model Study of Scour Around Bridge Piers and Abutments" by E.M. Laursen and A. Toch (3) has yielded additional information for scour depths. This study is similar to the Hardinge Bridge pier analysis by Inglis. The authors have brought forward a concept of active scour and indicate that the depth of flow might be involved in determining depths of scour holes. Their Figure 10 page 129 was used to obtain values for depth of scour measured from water surface and discharge intensity in order to plot on the graph $d_s F_b^{1/3}$ vs q; the calculations and results are recorded in Table 5.

E.M. Laursen (4) made a special model of an actual prototype. He superimposed his model results on data obtained by Hubbard from the prototype. Close relationship was found between the model and prototype at the lower values of discharge. For this particular case only the observations by Laursen were used in the writer's plot. The calculations are shown in Table 5.

A study by Mushtaq Ahmad entitled "Experiments on Design and Behavior of Spur Dikes" (3) had two figures 3 (a) and 4 (a) indicating slow asymptotic increase in depth of scour with time. How quickly the ultimate depth was reached depended mainly upon discharge intensity and grain size of the bed. This factor should be kept in mind when determining scour by models. Table 6 has the computations leading to values for the $d_s F_b^{y_3}$ vs q plot from figures 3 (a) and 4 (a), where the d_s values were the ultimate depths obtained for each discharge.

A University of Alberta Civil Engineering 66 Laboratory report by R. Sanders and S. Smotrych listed several scour depths. These values are shown in Table 7 where d_s is the observed depth and q is the total discharge available divided by the model distance between guide banks.

The last table of data, Table 8 for Part I, was collected while both river trays were being prepared to obtain regime conditions, for other parts of this report. This preparation resulted in continous flow for approximately fourteen days. Fairly stable conditions were obtained in the east river tray with the fine sand and very stable conditions resulted in the west tray with its coarse sand.

All the values of q in Table 8 were estimated by the width of the main flow attacking the structure and the percentage of total discharge of 0.11 cusecs. in the flow. Both factors are difficult to estimate especially obtaining the discharge intensity.

RESULTS

The initial graph, termed the "master plot", had all the data in Tables I to 8 plotted to the coordinates $d_s F_b^{1/3}$ vs q (Fig. 1). The best fit line by eye through the points has a slope of 0.74 . The equation then is $d_s F_b^{1/3} = 1.35\ q^{0.74}$. According to the regime theory the discharge intensity index should be two-thirds for canal - like conditions. Setting this slope on the master plot and fitting it to pass through the field data gives an equation, $d_s F_b^{1/3} = 1.8\ q^{2/3}$, which passes through the higher model points. As the writer's work shows that scour in some models may not have reached its full depth, and as the flume models gave the q actually attacking obstacles while the rivers gave average q of whole river width model points must be expected to be biased to the right. Therefore the regime line is as plausible as the "best fit" one.

The master plot was next divided into three. These divisions are bridge pier scour, scour at obstructions, and scour downstream of bridge piers. In Fig. 2 the results of bridge pier data show about the same difference as in Fig. 1 between obtaining scour depth by the best fit line of equation $d_s F_b^{1/3} = 1.4\ q^{0.7}$ and the regime equation of $d_s F_b^{1/3} = 1.7\ q^{2/3}$.

Fig. 3 for guide banks, groynes, spurs and sharp bends is not much different from Fig. 2 . One line slopes at 0.7 and the other at 2/3. The multiplying factor for the former is 1.7 and the latter is 2.05 times "regime depth". The correct

determination hinges upon choosing a correct discharge intensity. For data of Table 6 q was the flow per unit width in the constricted portion of the channel. For attack on guide banks and groynes q resulted from considering the amount of flow concentrating on the obstruction and its width.

Fig. 4 for scour downstream of bridges had insufficient points to fix a line. Lines of slope 2/3 have been drawn through different groups of data to obtain regime depth multipliers. The Fraser River data were not spectacular although associated with a bridge failure.

CONCLUSIONS

1. The accumulation of more field data or model data from a large outdoor hydraulic station is needed to fill the gap between 5 cusecs./ft. to 30 cusecs./ft.

2. The regime relation that depth of a channel varies as discharge intensity to the power two-thirds, all other factors being equal, seems confirmed reasonably after allowing for the differences in measuring q.

3. For severe scour at bridge piers an average relation is $d_s = 1.7\sqrt[3]{q^2/F_b}$, q being the mean discharge per unit width upstream of the bridge waterway.

4. For severe scour at obstructions an average relation is $d_s = 2.05\sqrt[3]{q^2/F_b}$, with q having the following qualifications (a) guide walls - q measured as the percentage of water attacking divided by the width of attack. (b) spurs - q measured in the ~~un~~confined portion of flow.

5. For scour downstream of bridge piers, there are insufficient data, and q is difficult to estimate because waterways may have been blocked.

6. The values of multipliers in the above findings do not justify any change in design coefficients already on record (1), (6).

TABLE I

Bridge Pier Scour From Table 8-I (1)

(See Appendix A-I for original data)

Site	Q cusecs.	d_s ft.	m (mm)	$F_b^{1/3}$ $=(1.9\sqrt{m})^{1/3}$	$q=\sqrt{Q/5}$ cusecs/ft.	$d_s F_b^{1/3}$	$d_r=q^{2/3}/F_b^{1/3}$ ft.	$\frac{d_s}{d_r}$
1(a)	1,675,000	117	0.37	1.05	579	123	66.6	1.76
2	2,250,000	104	0.39	1.06	671	110	73.5	1.42
3	800,000	94	0.32	1.03	400	96	52.5	1.79
4	309,000	57	0.32	1.03	249	58	35.9	1.59
5	43,000	38	0.32	1.03	93	38	19.9	1.88
6	300,000	74	0.38	1.05	245	78	35.2	2.10
7	168,000	58	0.33	1.03	183	60	31.2	1.88
8	414,000	83	0.17	0.92	288	76	47.4	1.75
9	29,063	25	0.34	1.03	76	26	17.5	1.43
10	60,207	48	0.32	1.03	110	49	22.3	2.15
11	103,115	42	0.37	1.05	144	44	26.2	1.59
12	109,834	41	0.30	1.01	148	42	27.7	1.50
13	156,021	45	0.34	1.03	177	46	30.6	1.47
14	167,357	65	0.34	1.03	183	68	31.2	2.09
15	69,747	41	0.24	0.98	118	40	24.5	1.65
16	61,314	39	0.32	1.03	111	40	22.3	1.75
17	106,747	49	0.20	0.95	146	46	29.5	1.66

TABLE 2

Scour at Guide Banks, Groynes, and Sharp Bends

From Table 8-2 (1)

(See Appendix A-2 for original data)

Site	Q cusecs	d_s ft	m (mm)	$F_b^{1/3}$	$q=\sqrt{Q/5}$ cusecs/ft.	$d_s F_b^{1/3}$	d_r ft.	$\frac{d_s}{d_r}$
1(b)	1,700,000	206	0.39	1.06	583	218	66.3	3.10
1(c)	1,700,000	150	0.37	1.05	583	158	66.7	2.25
2	50,000	66	0.27	1.00	100	66	21.6	3.06
3	1,000,000	106	0.32	1.03	447	109	56.5	1.88
4	300,000	60	0.27	1.00	245	60	39.2	1.53
5(b)	48,000	30	0.32	1.03	98	31	20.8	1.44
(c)	31,000	24	0.32	1.03	79	25	18.0	1.33
6(a)	209,883	65	0.32	1.03	205	66	34.1	1.90
(b)	209,883	68	0.32	1.03	205	70	34.1	2.00
7(a)	50,700	65	0.32	1.03	101	66	21.2	3.06
(b)	40,000	56.4	0.32	1.03	89	58	21.0	2.68

TABLE 3

Scour Downstream of Bridges

From Table 8-3 (1)

(See Appendix A-3 for original data)

Site	Q cusecs	d_s ft.	m (mm)	$F_b^{1/3}$	$q=\sqrt{Q/5}$ cusecs/ft.	$d_s F_b^{1/3}$	d_r ft.	$\frac{d_s}{d_r}$
1 (d)	1,670,000	216	0.37	1.05	579	227	66.6	3.24
2	10,900	47	0.07	0.79	47	37	16.4	2.87
3	10,900	56	0.06	0.77	47	43	16.9	3.31
4	5,000	31.5	0.30	1.01	32	32	9.9	3.18
5	10,050	45	0.32	1.03	45	46	12.3	3.66

TABLE 4

Scour at Hardinge Bridge Pier Models Unprotected

From Figure 8-6

(See Appendix A-4 for original data)

b = 37' prototype

Model Scale 1/65 b = 0.57'

-m (mm)	$F_b^{1/3}$	d_s/b	d_s ft.	$q_c^{2/3}/b$	q_c cusecs/ft.	$d_s F_b^{1/3}$	d_r ft.	$\frac{d_s}{d_r}$
0.29	1.01	3.10	1.76	1.98	1.20	1.78	1.12	1.57
		3.55	2.02	2.40	1.61	2.04	1.36	1.49
		3.95	2.25	3.10	2.35	2.27	1.75	1.29
		4.70	2.68	3.22	2.49	2.70	1.82	1.47
		4.78	2.72	3.42	2.71	2.74	1.93	1.41
		4.79	2.73	3.58	2.92	2.76	2.02	1.35
Model Scale 1/105 b = 0.35'								
0.29	1.01	3.00	1.06	2.42	.78	1.07	.84	1.26
		4.00	1.41	3.25	1.22	1.42	1.13	1.25
		5.00	1.75	4.00	1.68	1.78	1.40	1.26
Model Scale 1/210 b = 0.18'								
0.29	1.01	2.95	0.52	2.05	.22	.52	.36	1.44
		3.75	0.66	3.05	.39	.66	.53	1.25
		4.65	0.82	3.80	.55	.83	.66	1.24
		5.90	1.04	4.80	.78	1.05	.84	1.24
		6.45	1.14	6.30	1.17	1.14	1.10	1.04
		8.20	1.44	7.60	1.55	1.45	1.33	1.08
		10.00	1.76	10.00	2.34	1.78	1.74	1.01
Model Scale 1/65 b = 0.57'								
1.3	1.29	2.80	1.60	1.90	1.13	2.06	.84	1.90
		3.05	1.74	2.23	1.43	2.25	.98	1.78
		3.87	2.21	2.75	1.96	2.86	1.22	1.81
		4.15	2.36	3.28	2.56	3.06	1.45	1.63

TABLE 5

Scour around Bridge Piers (model)

From Figure 10 (3)

(See Appendix A-5 for original information)

m (mm)	$F_D^{1/3}$	d_s plus D ft.	Depth of flow D in ft.	V ft/sec.	q = VD cusecs/ft.	$d_s F_D^{1/3}$	d_r ft.	$\frac{d_s}{d_r}$
0.58	1.13	0.57	0.2	1.00	0.20	0.64	.30	1.89
1.60	1.34	0.57		1.75	0.35	0.76	.37	1.54
0.58	1.13	0.57		1.25	0.25	0.64	.35	1.62
0.58	1.13	0.55		1.50	0.30	0.62	.40	1.39
1.60	1.34	0.54		2.00	0.40	0.72	.41	1.33
2.20	1.41	0.73	0.3	2.25	0.68	1.03	.55	1.34
2.20	1.41	0.72		2.50	0.75	1.02	.58	1.23
1.60	1.34	0.70		2.25	0.68	0.94	.58	1.21
0.58	1.13	0.69		1.25	0.38	0.78	.46	1.49
1.60	1.34	0.68		2.00	0.60	0.91	.53	1.28
0.58	1.13	0.67		1.50	0.45	0.76	.52	1.29
0.58	1.13	0.67		2.50	0.75	0.76	.73	0.92
0.58	1.13	0.67		2.00	0.60	0.76	.63	1.06
0.46	1.09	0.67		1.25	0.38	0.73	.48	1.40
1.60	1.34	0.92	0.45	1.75	0.79	1.23	.64	1.44
1.60	1.34	0.92		2.00	0.90	1.23	.70	1.32
2.20	1.41	0.90		2.50	1.12	1.27	.76	1.18
1.60	1.34	0.90		2.25	1.01	1.20	.75	1.20
2.20	1.41	1.08	0.60	2.50	1.50	1.52	.93	1.16
1.60	1.34	1.07		2.00	1.20	1.43	.84	1.27
0.58	1.13	1.07		1.75	1.05	1.21	.92	1.17
0.46	1.09	1.05		1.25	0.75	1.14	.75	1.39
0.58	1.13	1.05		1.50	0.90	1.19	.82	1.28
0.58	1.13	1.05		2.00	1.20	1.19	1.00	1.05
1.60	1.34	1.29	0.80	2.25	1.80	1.72	1.11	1.16
2.20	1.41	1.29		2.50	2.00	1.82	1.13	1.14
0.58	1.13	1.52	0.90	1.25	1.12	1.72	.96	1.58
0.58	1.13	1.45		1.50	1.35	1.64	1.08	1.34
0.58	1.13	1.43		2.00	1.80	1.62	1.31	1.07
0.58	1.13	1.40		1.75	1.58	1.58	1.20	1.17
FROM Figure 5 (4)								
0.58	1.13	0.55	0.2	1.75	0.35	0.62	0.44	1.25
		0.68	0.3		0.52	0.77	0.58	1.18
		1.00	0.6		1.05	1.13	0.92	1.09
		1.17	0.75		1.19	1.33	0.99	1.19
		1.29	0.85		1.35	1.46	1.08	1.20
		1.35	0.90		1.43	1.52	1.12	1.21
		1.44	0.94		1.49	1.62	1.15	1.25

TABLE 6

Scour around Spur Noses

From Figure 3 (a)

(see Appendix A-6 for original data)

m (mm)	$Fb^{1/3}$	d_s ft.	q cusecs/ft.	$d_s Fb^{1/3}$	d_r ft	$\frac{d_s}{d_r}$
0.25	0.983	1.10	0.667	1.08	.77	1.43
		0.93	0.512	.92	.65	1.43
		0.74	0.362	.73	.52	1.42
		0.54	0.235	.53	.39	1.42
		0.39	0.130	.38	.26	1.33

From Figure 4 (a) Scour around Spur Noses
(see Appendix A-6 for original data)

0.354	1.04	0.960	0.3396	1.00	.46	2.09
		0.864	0.2828	0.90	.41	2.10
		0.710	0.2263	0.74	.36	2.00
		0.560	0.196	0.58	.33	1.72
0.696	1.17	0.960	0.3396	1.12	.41	2.34
		0.864	0.2828	1.01	.37	2.34
		0.710	0.2263	0.83	.32	2.22
		0.560	0.196	0.65	.29	1.92

TABLE 7

From C.E. 66 Lab. Report (7)

Noses of guide banks

m (mm)	$F_b^{1/3}$	d_s ft.	q cusecs/ft.	$d_s F_b^{1/3}$	d_r ft.	$\frac{d_s}{d_r}$	Comments
0.27	1.00	0.33	.094	.33	0.20	1.65	
		0.25		.25		1.25	
		0.46		.46		2.30	severe attack
		0.47		.47		2.35	severe attack
		0.32		.32		1.60	
Along guide banks							
0.27	1.00	0.35	0.094	.35	0.20	1.75	
		0.31		.31		1.55	

TABLE 8

University of Alberta Data - Andru

East River Tray

Q cusecs	d_s ft.	m (mm)	$F_b^{1/3}$	q $\frac{\text{cusecs}}{\text{ft.}}$	$d_s F_b^{1/3}$	d_r ft.	$\frac{d_s}{d_r}$	Comments
0.11	0.11	0.27	1.	.022	0.11	.078	1.41	along guide bank (not severe)
	0.15			.066	0.15	.163	0.92	meander bend
	0.24			.094	0.24	.206	1.16	along guide bank
	0.16			.022	0.16	.078	2.05	along guide bank
	0.12			.066	0.12	.163	.74	meander bend
	0.15			.088	0.15	.198	.76	along guide bank
	0.13			.066	0.13	.163	.80	along guide bank
	0.35			.167	0.35	.303	1.15	along guide bank
	0.13			.073	0.13	.175	.74	along guide bank
	0.15			.022	0.15	.078	1.92	along guide bank
	0.12			.077	0.12	.181	0.66	along guide bank
	0.35			0.11	0.35	.23	1.52	along guide bank
	0.12			0.15	0.12	.282	.43	along guide bank
	0.14			.044	0.14	.124	1.13	along guide bank
	0.11			.038	0.11	.113	.97	along guide bank
	0.20			.099	0.20	.214	.93	along guide bank
	0.15			.099	0.15	.214	.70	along guide bank
	0.21			.05	0.21	.136	1.55	sharp bend
	0.12			.049	0.12	.134	.90	meander bend
	0.38			.099	0.38	.214	1.78	upstream spur nose
	0.18			.099	0.18	.214	.84	along spur wall
	0.18			.099	0.18	.214	.84	downstream spur nose
	0.27			.099	0.27	.214	1.26	upstream spur nose

TABLE 8 (cont'd)

Q cusecs	d_s ft.	m (mm)	$F_b^{1/3}$	q cusecs/ft.	$d_s F_b^{1/3}$	d_r ft.	$\frac{d_s}{d_r}$	Comments
0.11	0.27	0.27	1	0.66	0.27	.163	1.66	along guide bank
	0.15			.033	0.15	.103	1.46	along guide bank
	0.23			.077	0.23	.181	1.27	sharp bend
	0.40			.099	0.40	.214	1.87	upstream spur nose
	0.15			.099	0.15	.214	.70	downstream spur nose
	0.17			.099	0.17	.214	.80	along spur wall
	0.28			.099	0.28	.214	1.31	upstream spur nose
	0.15			.033	0.15	.103	1.46	along guide wall
	0.23			.077	0.23	.181	1.27	sharp bend
	0.40			.099	0.40	.214	1.87	upstream spur nose
	0.17			.099	0.17	.214	.80	along spur wall
	0.15			.099	0.15	0.214	.70	downstream spur nose
	0.28			.099	0.28	.214	1.31	upstream spur nose
	0.15			.011	0.15	.05	3.00	along guide wall
	0.22			.099	0.22	.214	1.03	along guide wall
	0.15			.077	0.15	.181	.83	along guide wall
	0.20			.077	0.20	.181	1.10	along guide wall
	0.35			.099	0.35	.214	1.64	upstream spur nose
	0.12			.099	0.12	.214	.56	downstream spur nose
	0.10			.099	0.10	.214	.47	along spur wall
	0.25			.099	0.25	.214	1.17	upstream spur nose
	0.13			.011	0.13	.05	2.60	along guide wall
	0.15			.099	0.15	.214	.70	along guide wall
	0.18			.066	0.18	.163	1.10	along guide wall
	0.20			.077	0.20	.181	1.10	sharp bend
	0.30			.099	0.30	.214	1.40	upstream spur nose
	0.18			.099	0.18	.214	.84	along guide wall

TABLE 8 (cont'd.)

Q cusecs	d_s ft.	m (mm)	$F_b^{1/3}$	q cusecs/ft.	$d_s F_b^{1/3}$	d_r ft.	$\frac{d_s}{d_r}$	Comments
0.11	0.15	0.27	1	0.066	0.15	0.163	0.92	along guide wall
	0.22			0.077	0.22	0.181	1.22	sharp bend
	0.35			0.088	0.35	0.198	1.77	upstream spur nose
	0.20			0.088	0.20	0.198	1.01	downstream spur nose
	0.20			0.088	0.20	0.198	1.01	along spur wall
	0.23			0.088	0.23	0.198	1.16	upstream spur nose
	0.15			0.022	0.15	0.078	1.92	along guide wall
	0.30			0.077	0.30	0.181	1.66	sharp bend
	0.35			0.077	0.35	0.181	1.93	upstream spur nose
	0.15			0.077	0.15	0.181	0.83	downstream spur nose
	0.15			0.077	0.15	0.181	0.83	along spur wall
	0.30			0.088	0.30	0.198	1.52	upstream spur nose
	0.17			0.088	0.17	0.198	0.86	along guide wall
	0.20			0.077	0.20	0.181	1.10	sharp bend
	0.10			0.077	0.10	0.181	0.55	downstream spur nose
	0.10			0.077	0.10	0.181	0.55	along spur wall
	0.30			0.132	0.30	0.259	1.16	upstream spur nose
	0.15			0.099	0.15	0.214	0.70	along guide wall
	0.15			0.082	0.15	0.19	0.79	along guide wall
	0.30			0.082	0.30	0.19	1.58	sharp bend
	0.15			0.088	0.15	0.198	0.76	along spur wall
	0.15			0.088	0.15	0.198	0.76	downstream spur nose
	0.30			0.099	0.30	0.214	1.40	upstream spur nose
	0.13			0.088	0.13	0.198	0.66	along guide wall
	0.16			0.07	0.16	0.17	0.94	bridge pier unprotected

TABLE 8 (Cont'd)

Q cusecs	ds ft.	m mm.	$F_b^{1/3}$	q cusecs/ft	$dsF_b^{1/3}$	dr ft.	$\frac{ds}{dr}$	Comments
0.11	0.20	0.27	1	0.07	0.20	0.17	1.18	bridge pier unprotected
	0.23			0.07	0.23	0.17	1.35	bridge pier unprotected
	0.19			0.07	0.19	0.17	1.12	bridge pier unprotected
	0.19			0.058	0.19	0.15	1.27	bridge pier unprotected
	0.17			0.058	0.17	0.15	1.13	bridge pier unprotected
	0.14			0.058	0.14	0.15	0.93	bridge pier unprotected
	0.15			0.058	0.15	0.15	1.00	bridge pier unprotected
	0.11			0.058	0.11	0.15	0.73	bridge pier unprotected
	0.18			0.058	0.18	0.15	1.2	bridge pier unprotected
	0.14			0.058	0.14	0.15	0.93	downstream of bridge piers unprotected
	0.17			0.058	0.17	0.15	1.13	downstream of bridge piers unprotected
	0.18			0.058	0.18	0.15	1.20	downstream of bridge piers unprotected
	0.15			0.058	0.15	0.15	1.0	downstream of bridge piers unprotected
	0.16			0.058	0.16	0.15	1.07	downstream of bridge piers unprotected
	0.15			0.07	0.15	0.17	0.88	downstream of bridge piers unprotected
	0.13			0.07	0.13	0.17	0.76	downstream of bridge piers unprotected
	0.12			0.07	0.12	0.17	0.71	downstream of bridge piers unprotected
	0.14			0.07	0.14	0.17	0.82	downstream of bridge piers unprotected

TABLE 8 (cont'd)

West River Tray

Q cusecs	ds ft.	m mm.	$F_b^{1/3}$	q cusecs/ft	$dsF_b^{1/3}$	dr ft.	$\frac{ds}{dr}$	Comments
0.11	0.19	0.41	1.07	0.050	0.20	0.127	1.50	bridge piers
	0.33			0.07	0.35	0.165	0.20	spur nose
	0.27			0.055	0.29	0.135	2.0	along guide wall
	0.17			0.050	0.18	0.127	1.34	bridge piers
	0.34			0.074	0.36	0.165	2.06	spur nose
	0.15			0.05	0.16	0.127	1.18	brider piers
	0.28			0.055	0.30	0.135	2.07	along guide wall
	0.17			0.047	0.18	0.121	1.40	downstream bridge piers protected
	0.13			0.047	0.14	0.121	1.07	downstream bridge piers protected
	0.15			0.047	0.16	0.121	1.24	downstream bridge piers protected

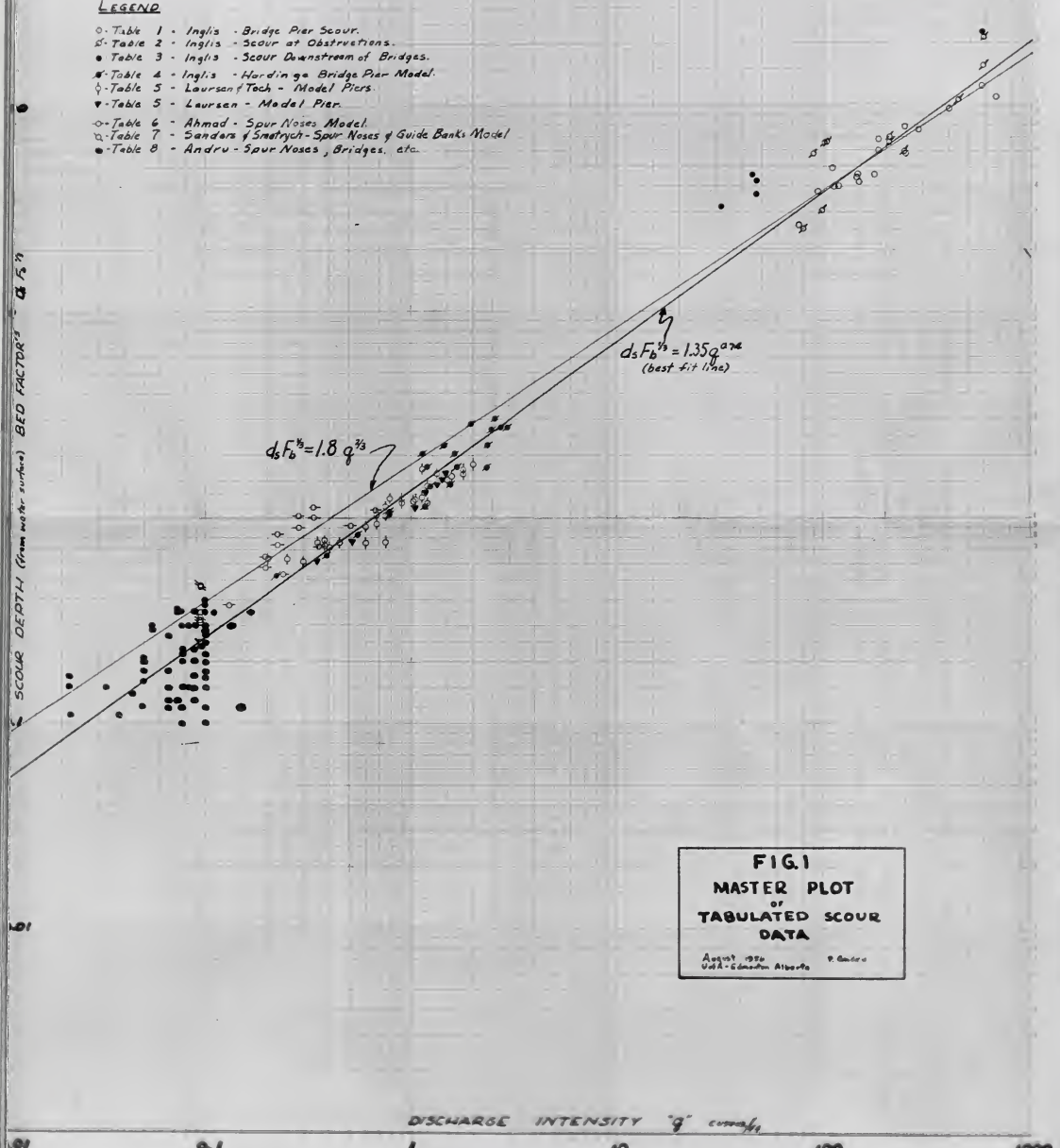

LEGEND
Table 1 - Inglis - Bridge Pier Scour.
Table 2 - Inglis - Scour at Obstructions.
Table 3 - Inglis - Scour Downstream of Bridges.
Table 4 - Inglis - Hardinge Bridge Pier Model.
Table 5 - Laursen & Toch - Model Piers.
Table 6 - Ahmad - Spur Noses Model.
Table 7 - Sanders & Smetzych - Spur Noses & Guide Banks Model
Table 8 - Andru - Spur Noses, Bridges, etc.
$d_s F_b^{1/3} = 1.35 q^{0.74}$
(best fit line)
$d_s F_b^{1/3} = 1.8\, q^{2/3}$
SCOUR DEPTH (from water surface) BED FACTOR$^{1/3}$ = $d_s F_b^{1/3}$
DISCHARGE INTENSITY "q" cumec/ft
.01
0.1
1
10
100
1000

FIG. 1
MASTER PLOT
OF
TABULATED SCOUR DATA

August 1970
U of A - Edmonton Alberta
P. Sanders

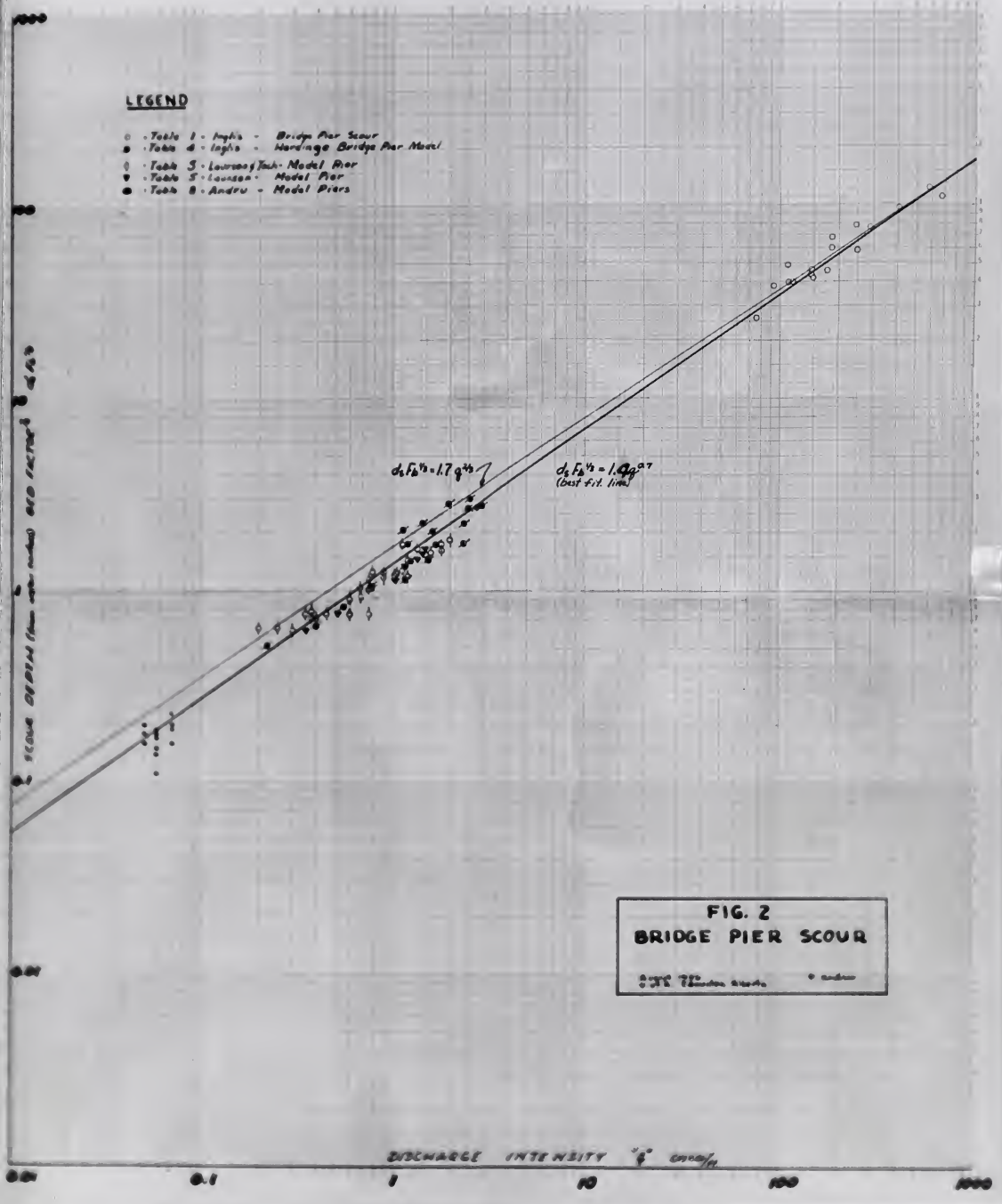


FIG. 2
BRIDGE PIER SCOUR

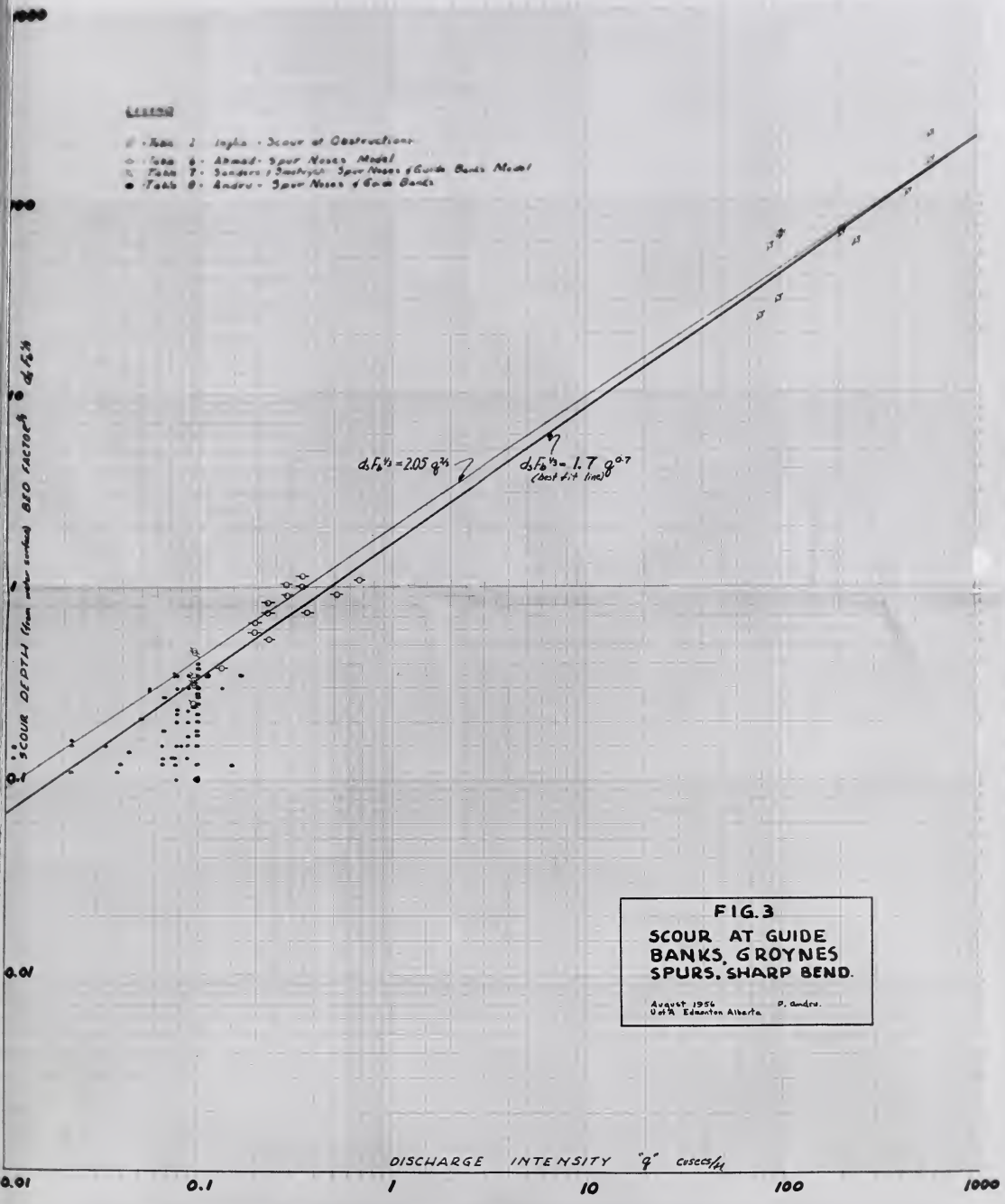

LEGEND
Tests 1 - Inglis - Scour at Obstructions
Tests 6 - Ahmad - Spur Nose, Model
Tests 7 - Sanders / Smith - Spur Noses of Guide Banks, Model
Tests 8 - Andru - Spur Noses of Guide Banks
$d_s F_d^{1/3} = 2.05\, q^{2/3}$
$d_s F_d^{1/3} = 1.7\, q^{0.7}$
(best fit line)
SCOUR DEPTH (from undisturbed BED FACTOR $d_s F_d^{1/3}$
DISCHARGE INTENSITY "q" cusecs/ft
0.01
0.1
1
10
100
1000

FIG. 3
SCOUR AT GUIDE BANKS, GROYNES SPURS, SHARP BEND.

August 1954 U of A Edmonton Alberta P. Andru.

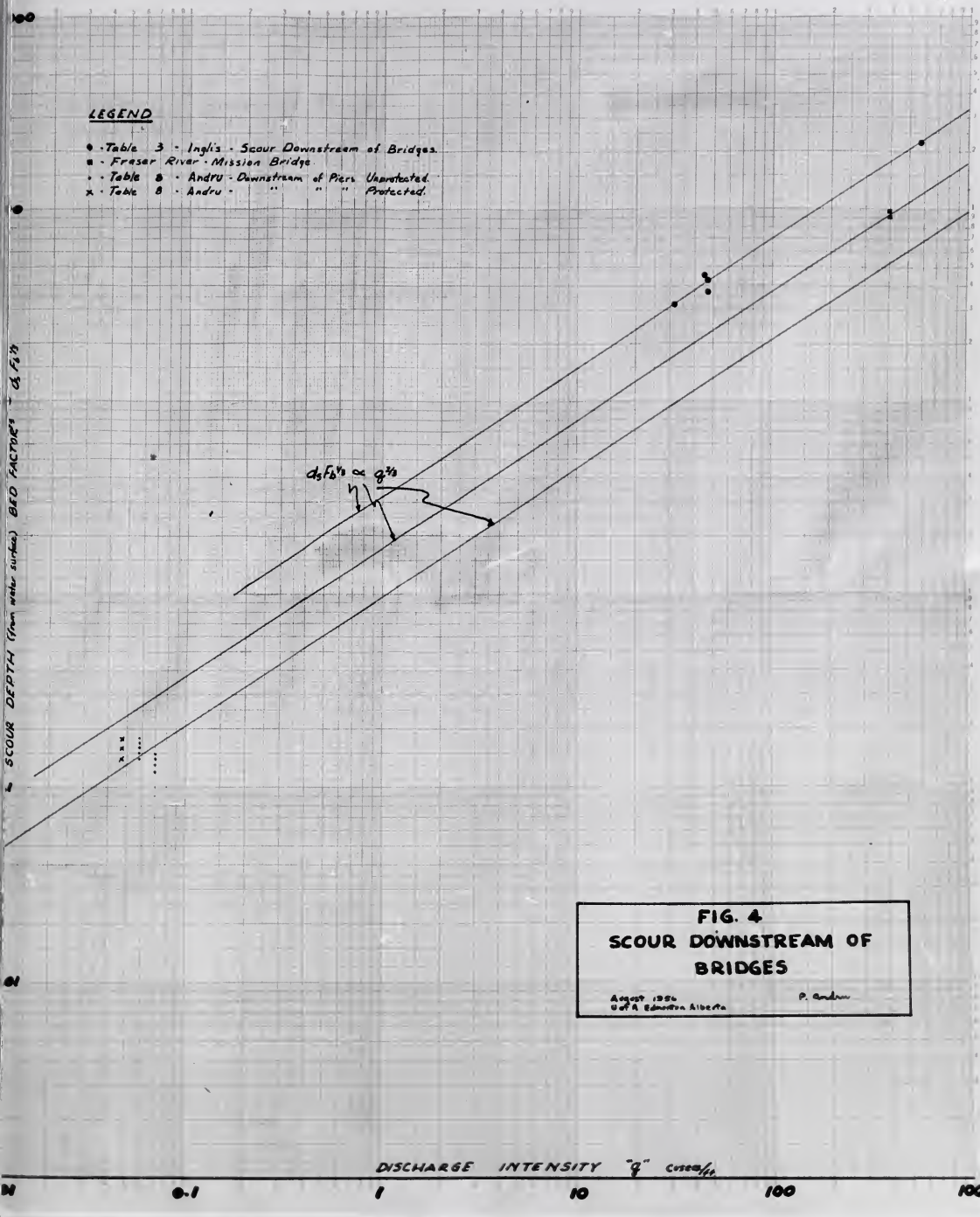

LEGEND
• - Table 3 - Inglis - Scour Downstream of Bridges.
■ - Fraser River - Mission Bridge
• - Table 8 - Andru - Downstream of Piers Unprotected.
x - Table 8 - Andru - " " " Protected.
$d_s F_b^{1/3} \propto q^{2/3}$
SCOUR DEPTH (from water surface) BED FACTOR$^{1/3}$ = $d_s F_b^{1/3}$
DISCHARGE INTENSITY "q" cusecs/ft
0.1
1
10
100
1000

FIG. 4
SCOUR DOWNSTREAM OF BRIDGES

August 1954
U of A Edmonton Alberta
P. Andru

P A R T II

SCOUR DOWNSTREAM OF PROTECTED BRIDGE PIERS

PART II

INTRODUCTION

Scour can be displaced, though not prevented, by proper placement of a non-cohesive material that cannot be transported or quickly destroyed by the water flow. Rock is often abundant and economical.

A model was constructed of a prototype where rock was placed around the piers periodically for many years. This placement had protected the piers through two major floods but a pier failed into the scour hole during a minor flood. The theory is that launched rock from adjacent piers met and formed a type of "V" notch weir. The effect was jetting action between the piers that caused scour downstream. Then failure of the pier occurred when the resulting scour hole undermined the stone around the structure.

The model, in spite of a variety of additions of stones, produced scouring action with two faint holes instead of the two conspicuous ones now present in the prototype. Apparently a barrage of stone does not cause severe scour ---- as shown by later experiments of this thesis ---- and the cause of the prototype scour was probably jetting from gaps in the barrage as in the theory.

EXPERIMENTAL PROCEDURE

Attainment of steady channel conditions.

The west river tray with its coarse sand bed of 0.41 mm was used for the model study. The channel was straightened as shown on Photo 1 with flow through the channel being continuous for a period of 432 hours. Water surface slopes on the centre line of the tray and scour depths at spur noses, along guide banks and bridge piers were measured with the scour values being tabulated for use in Part I. During this period the model piers were omitted. The long interval of time involved is due to the type of recirculating system used in the river tray construction. Any change in the bed of the channel along the tray took a very long time to return to its initial spot. One to two weeks seemed to be the time required to complete the cycle.

Determination of channel scales.

The vertical scale was determined as follows. Soundings measured from a geodetic datum on the prototype plan at a distance upstream of the bridge crossing were averaged. The value was 25 feet. To this figure must be added the height to high water, a value of 26 feet. Therefore the total depth, for flow of 600,000 cusecs, is 51 feet. The average depth of flow determined by the discharge of 0.11 cusecs over the area where the model was to be placed was 0.12 feet. Hence the vertical scale is 0.12 feet is equal to 51 feet or 1 foot equals 425 feet.

The horizontal scale was established by the distance between the parallel walls. These walls, placed 2.35 feet apart, were to mirror the prototype conditions of a straight reach ahead of the bridge. If the walls were closer together a depth of flow greater than 0.12 feet would result with a tighter arrangement of bridge piers. Turning to the other extreme, with the walls farther apart, a depth of flow would be obtained that measurements of scour depths around the bridge piers would be difficult. To model the total bridge waterway of the prototype case would have the piers too close together and each one would be affected by the other. Therefore a similar model bridge was constructed with less piers but including the large pier for the swing bay section. In this way the distance between piers would be a multiple factor of the depth. The distance 2.35 feet of the model would represent 1754 feet in the prototype and the horizontal scale is then 1 foot equals 746 feet.

These scales of 1 foot to 746 feet for the horizontal and 1 foot to 425 feet for the vertical result in the model having approximate geometric similarity with the prototype.

A quick check can be made to determine if the scales being used are reasonable.

By the equation $Q^2 = Fb\, b^2 d^3$, where Q is the discharge ratio, Fb the bed factor ratio, b the width ratio (horizontal scale) and d is the depth ratio(vertical scale), Fb the bed factor is calculated to be 1.43. This value of Fb is very reasonable, indicating that the scales chosen are satisfactory.

Model Layout and Dimensions

The bridge piers were constructed to the horizontal scale determined by the 2.35 foot width representing the total bridge length. The piers were spaced apart three to four times the model depth of flow of 0.12 feet. (See Plate 1 for dimensions and Photo 1 for location).

Discharge

A flow of 0.11 cusecs was used. The measurement of this discharge was by a "V" notch weir at the head of the tray. The operation of the river tray is explained in its entirety by Mr. A.W. Peterson(5).

Slope

The control exerted by the flap gate at the bottom of the tray adjusted the water surface slope to approximately a steady value of 0.07%.Actual slope measurements on the centre line of the tray were 0.14% and the regime slope by computation was 0.07%.

Apron Stone Size

The 1/8" stone had been used in another model where it had launched satisfactorily. Since the same discharge was used in both models, the same size of stone was placed. Scaling the stone size to the prototype dimensions results in 1/8" rock representing 4 foot rock of weight 6 tons. This factor indicates a smaller size of stone should be used in the model, but this new size is approaching the coarse sand sizes of the bed. Hence a median has to be decided and a size of stone placed determined by practical model experience.

Procedure

Since the rock was placed annually for the prototype case, the model was treated in the following way: the scaled rock would be placed for different scour depths, which were 0.5d, 0.75d and 1.0d measured below mean bed level where d was 0.12 feet. The first portion of 1/8" rock required was 3 layers, 0.75d wide by the length of the pier. Then for the other scour depths only 1.5 layers with the same dimensions would be placed. The initial 3 layers and the following 1.5 layers were quantities required to have the launched rock in 2 layers, with no special attention paid to the ends of the piers. These values were obtained by calculations as made on Data sheet 1.

For the first run of 0.5d scour, observations of the settlements were noted and a period of 7 days seemed satisfactory for obtaining equilibrium conditions in which final settlement had occurred. This period was used for the other scour depths as well. Complete sounding programs were made at the beginning and end of each run using a grid as shown in Photo 2. This grid was the fixed datum for all soundings taken. The height of the grid above water surface of 0.20 feet would be subtracted from the sounding recorded to obtain total depth to bed level. A correction for slope of the water surface under the grid was not made since the figure was very small and greater errors result from the sounding itself.

After each lapse of 7 days the recorded soundings were converted to prototype depths, using the scale 1 foot represents 425 feet. Contours were drawn for each case with photographs taken before and after each particular test run.

RESULTS

Run I --- 0.5d scour

Photo 3 shows the initial appearance (zero hours) of the area around the piers and the location of the stone. There was no attack initially without stone at the piers. The photograph indicates a deeper section to the left of the largest pier and observation of Plate 2 will give an idea of the depths prevailing. After 7 days the observations yielded a contour plan of final conditions for 0.5d scour (Plate 3). From Photo 4 large changes in the model sand bed have occurred. This is due to the obstructions to flow created by the piers. Comparing Photos 3 and 4, it can be clearly seen that in the shallower areas the stone has been covered by the sand. Faint scours have occurred downstream of the piers. They are crescent shaped and not like the two holes in the prototype. The depths of the scour holes that are closest to resembling the ones in the prototype are 55 feet and 47 feet.

Run II --- 0.75d scour.

The additional 1.5 layers of stone were placed around the piers. If the sand covered the original rock, no removal of the covering was made. This procedure would be similar to actual field conditions. The initial appearance is shown by Photo 5. It is identical to the final conditions of Run I Photo 4 except for the stone addition. In Photo 6 the crescent shaped scour holes have disappeared. Again sand has covered portions of the stone. Within the swingbay area the launched stone has met to form a complete obstruction between piers. Plate 4 shows the depths for final scour of 0.75d.

Run III --- 1.0d scour.

Photo 7 has the initial appearance for 1.0 d scour. After 504 hours the waterway between the piers is entirely covered with stone (Photo 8). Two scour holes (Plate 5) of depths 72 feet and 55 feet were obtained, but they are not within the area we are interested in.

Conditions similar to the prototype contour plan (Plate 6) we are trying to imitate were not obtained. Approach elevations were quite different, with an extremely deep section along the left side of the contour plans.

Run IV ---

The time factor was then considered as being important, and therefore the final conditions for Run III were subjected to an additional 310 hours. The model had then been under scouring conditions for a total time of 814 hours. After this test no scour holes have appeared.

Run V ---

The flow pattern was the next factor studied. As mentioned before there was a deep section along the left side. A study of the model cross-sections at the piers by comparing them with the similar prototype cross-section Plate 7, further assisted in pointing out that the water was tending to flow around the ends. This was probably due to the build up of sand and rock in the swing-bay area which tended to dam the flow and direct it around the ends. This condition led to the idea of raising the end elevations to that in the prototype, with no section above prototype elevation being removed. Photo 9 shows the additional rock in place at the ends. The

new elevations are practically the same as for the final conditions of Run III except at the ends where they are raised to prototype depth. (Plate 7 Run III).

Hardly any change had occurred in the area we were interested in, after an additional interval of 168 hours of flow (total to date 982 hours). The flow had a tendency to attack the swing bay section but the height of rock protection directs the discharge around the ends.

Run VI ---

The next step was to direct the flow at the swing bay area. This was done by placement of a spur (Photo 10). After a further 216 hours (1198 hours total) of attack, a slight hole was obtained, the deepest depth being 85 feet (0.20 ft. model). The water was still concentrating at the ends, but to a far lesser extent.

Run VII ---

More stone was added to bring the elevation higher than the prototype plan but in proportion the swing bay section was already above the prototype elevations. The appearance of the model bed between piers was now similar to the prototype bed with model bed being at a higher elevation. This state received 288 hours (total 1486 hours) of continuous discharge of 0.11 cusecs over the testing area. Two holes appeared, 38 feet and 55 feet deep. These values do not compare with the prototype case but the factor obtained from the ratio of depth of hole to depth of flow between piers is practically constant in both model and prototype. This factor is from 1.5 to 2.0. The final cross-section is drawn on plate 7

fig.(a). Photo 11 shows the two scour holes just behind the swing bay pier with the back edge of the hole against the pier. The size of the holes can be readily obtained from Plate 8.

SUMMARY

As in the prototype, stone was placed periodically at relatively long intervals. Initial placing had no appreciable effect and settlements were associated with the passage of bed waves. Continual observation of the erosion and scouring abilities of the flow in the coarse sand section of the river tray showed developments were very slow.

The model finally developed two scour holes after the launched rock had met. The holes were not of a spectacular size but they were there. Attempts to enlarge them by concentrating flow to one side of the bridge and adding stone in bays where the effect was not wanted did not produce enough jetting into the holes to increase their depth up to expectation from the prototype.

DATA SHEET I

I - To determine quantity of stone for 0.5d scour below mean bed level.

From bibliographies(1), Ahamd (3), the length L = 1.5 ds

Therefore length of level stone apron

L = 1.5 (0.5d) = 0.75d

length of inclined apron = $\sqrt{5}$ (0.5d) = 1.12d

2 layers (1.12d) = A layers (0.75d) where A is the number of layers for launching.

A = 2.24d/0.75d = 3.

The stone is placed in 3 layers 0.75d wide by the length of the pier.

II -Quantity of stone for scour of 0.75d below mean bed.

length of level apron 1.5(0.75d) = 1.125d

length of inclined apron is $\sqrt{5}$ (0.75d) = 1.68d

2 layers (1.68d) = A' layers (1.125d)

check A' layers = 3 layers.

Amount of stone placed and settled for 0.5d scour = 2.24d

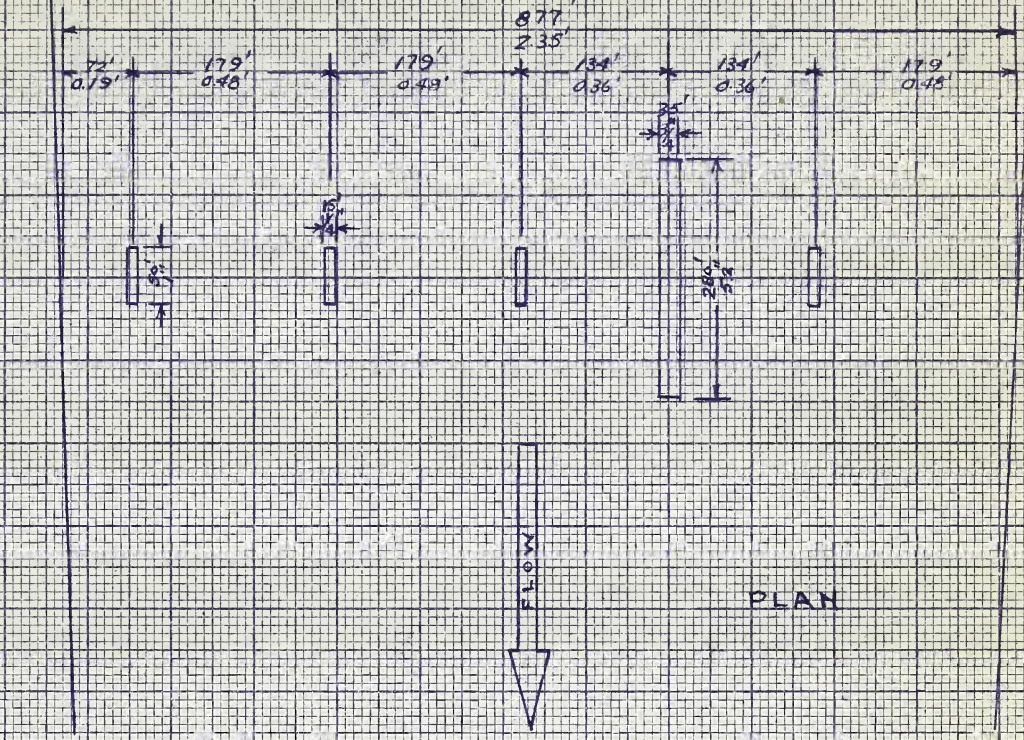
Amount of stone required for 0.75d scour = 3.36d

Amount of extra stone needed is 1.12d

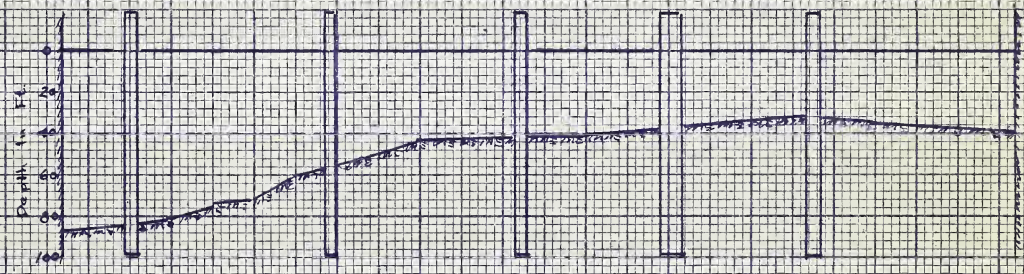
The new stone will be placed in the same area as for 0.5d scour. Therefore 1.12d = A'' layers (0.75d)

A'' = 1.5 layers

Therefore scours of 0.75d and 1.0d, additional amounts of 1.5 layers of stone placed in the area of 0.75d wide by the length of the pier are required.



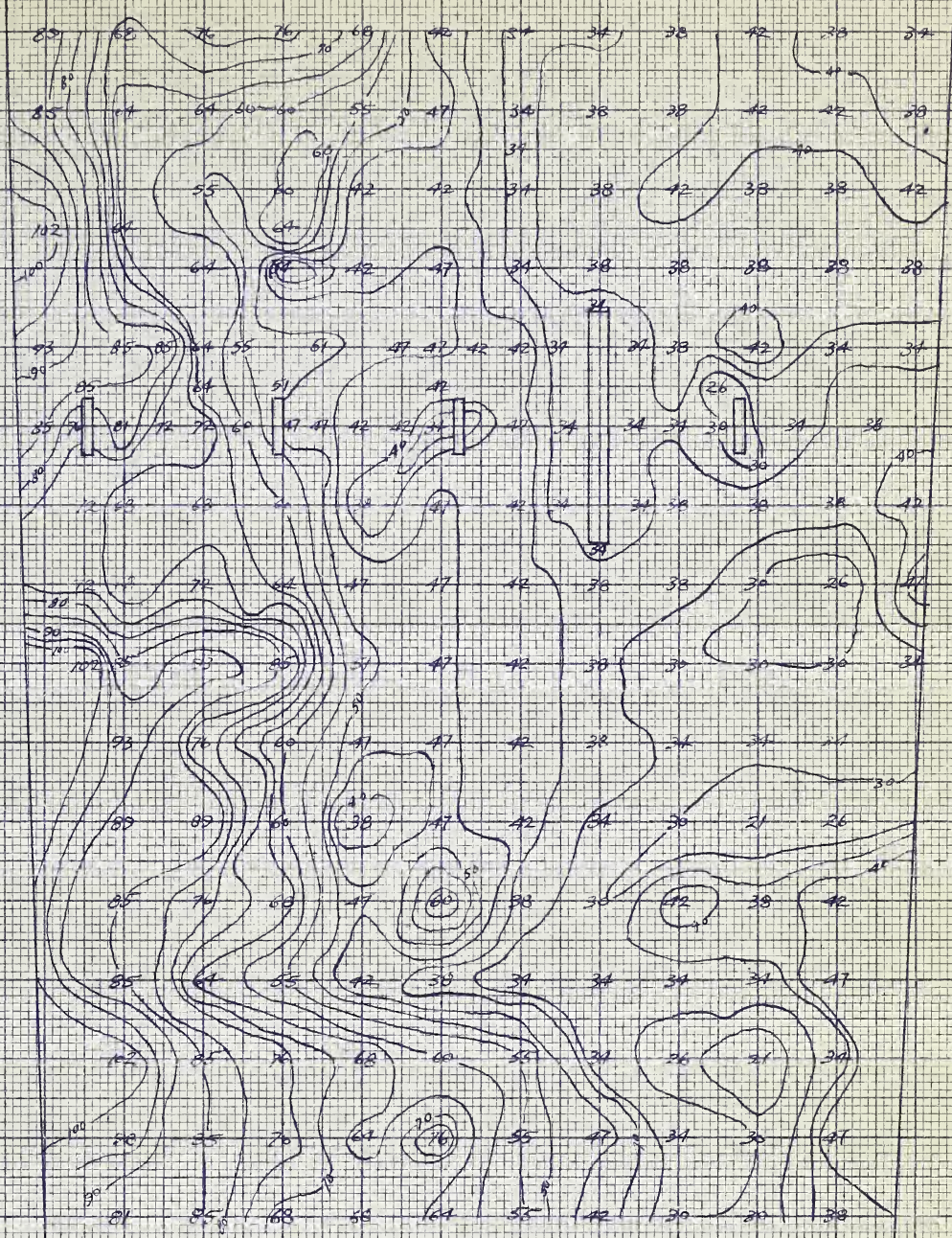
PLAN



ELEVATION
(model without stone protection)
(Time 0 hours)

MODEL LAYOUT
1954

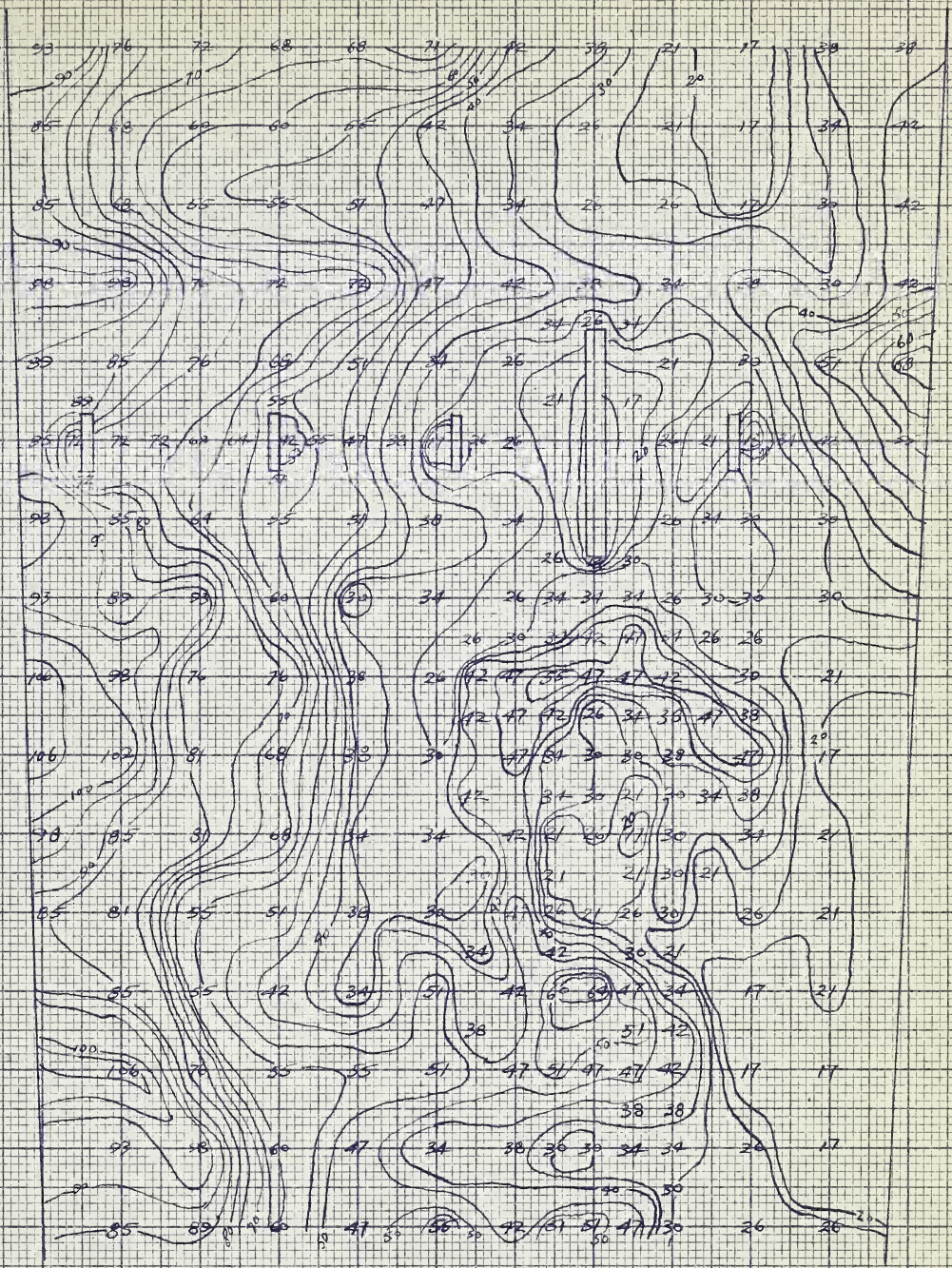
PLATE I



INITIAL DEPTHS RUN 1

CONTOUR INTERVAL 5'
G.E. OIL CABLES 1956

PLATE 2

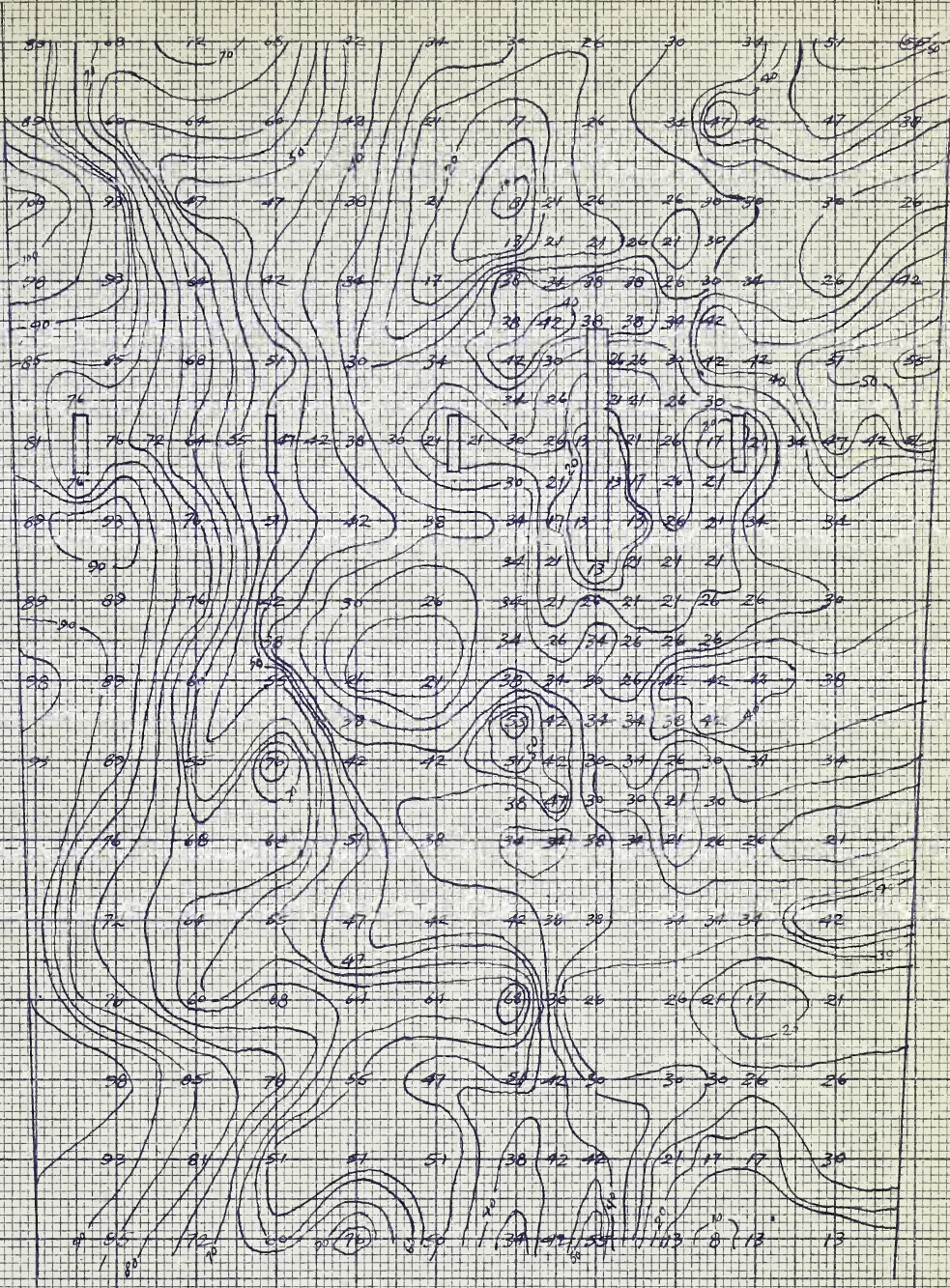


FINAL DEPTHS RUN I
Total Time 168 hrs.

INITIAL DEPTHS
RUN II

CONTOUR INTERVAL 5'
Q = 0.11 cucecs. 1956

PLATE 3

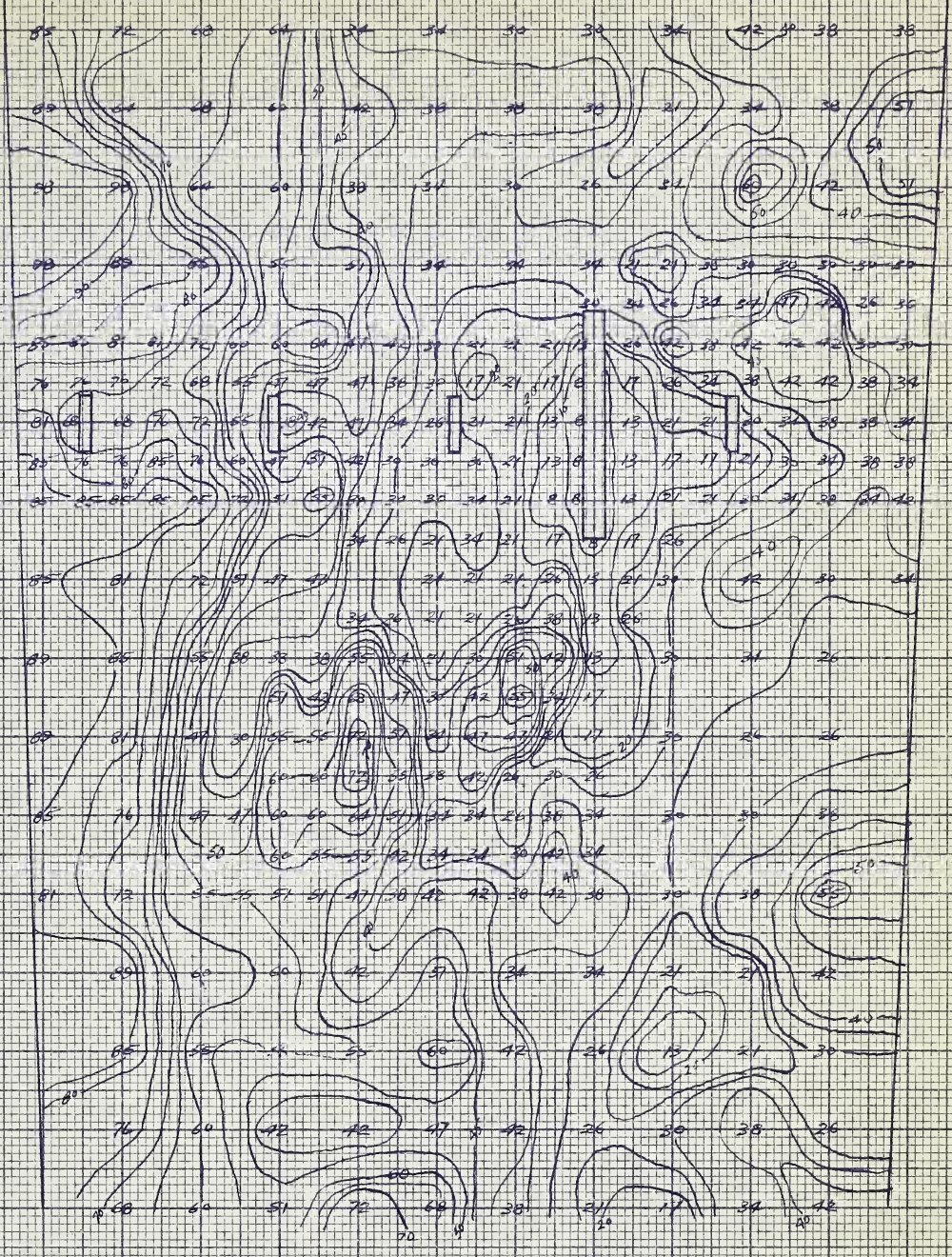


FINAL DEPTHS RUN III
Total Time 336 hrs

INITIAL DEPTHS
RUN III

CONTOUR INTERVAL 5'
SE Oil Co. 1956

PLATE 4

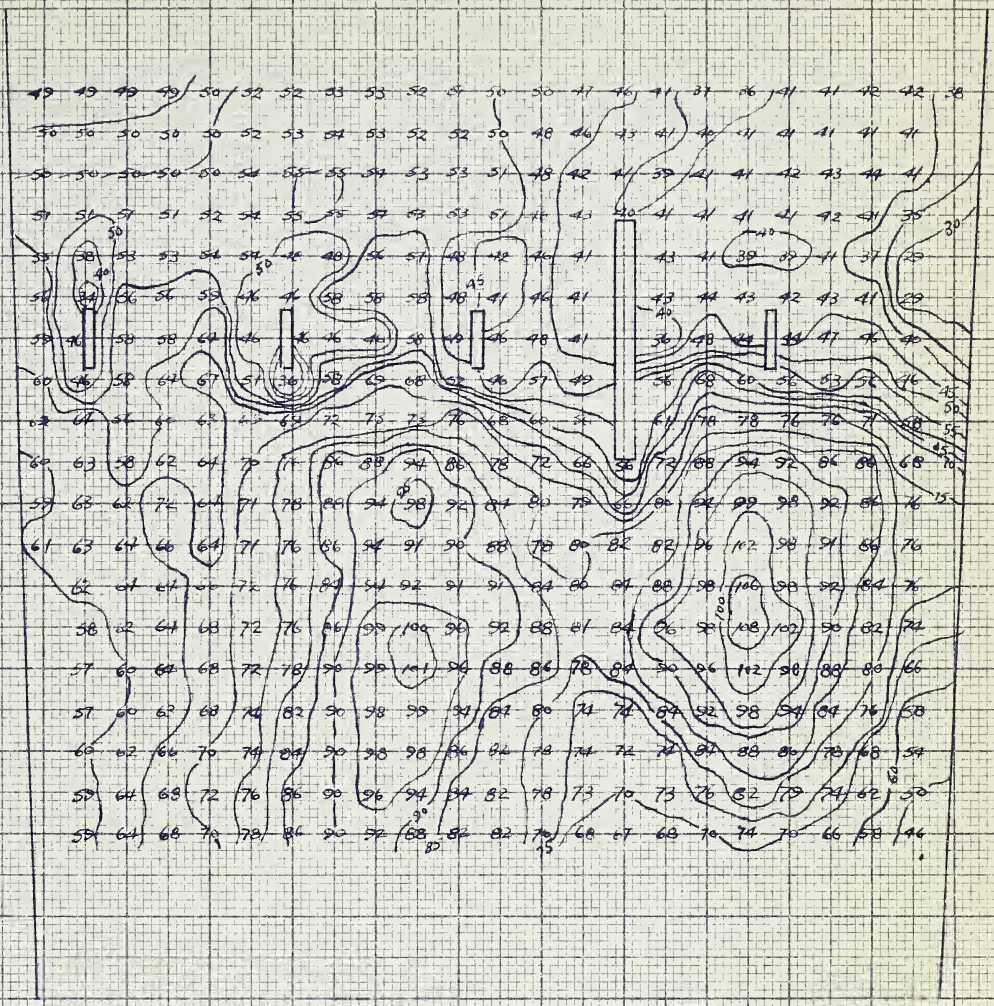


FINAL DEPTHS
RUN III

Total Time 504 hrs.

CONTOUR INTERVAL 5'
Q = 0.11 CUSECS. 1956

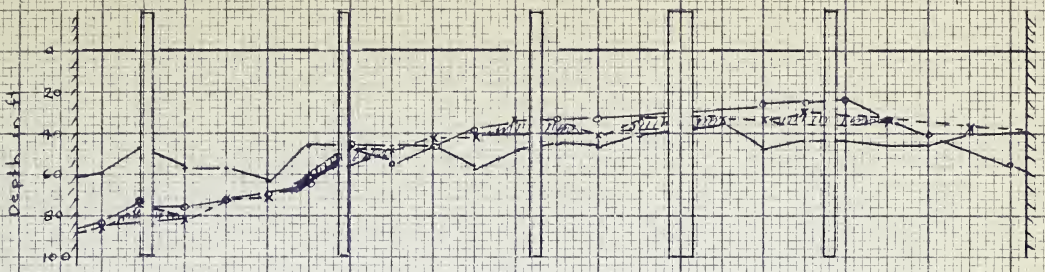
PLATE 5



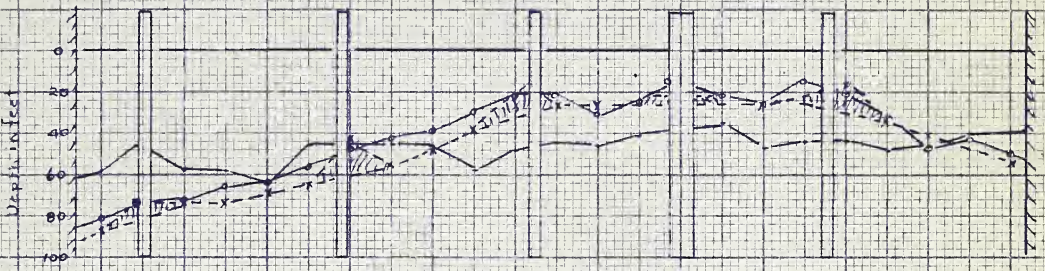
PROTOTYPE CONTOURS AND SCOUR HOLES

CONTOUR INTERVAL 5' 1955

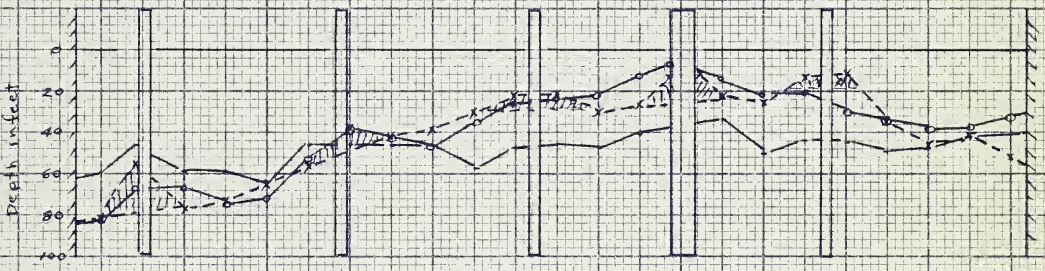
PLATE 6



Run I



Run II



Run III

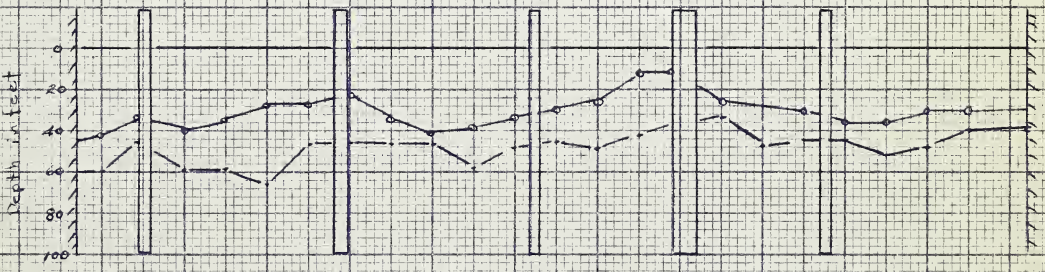
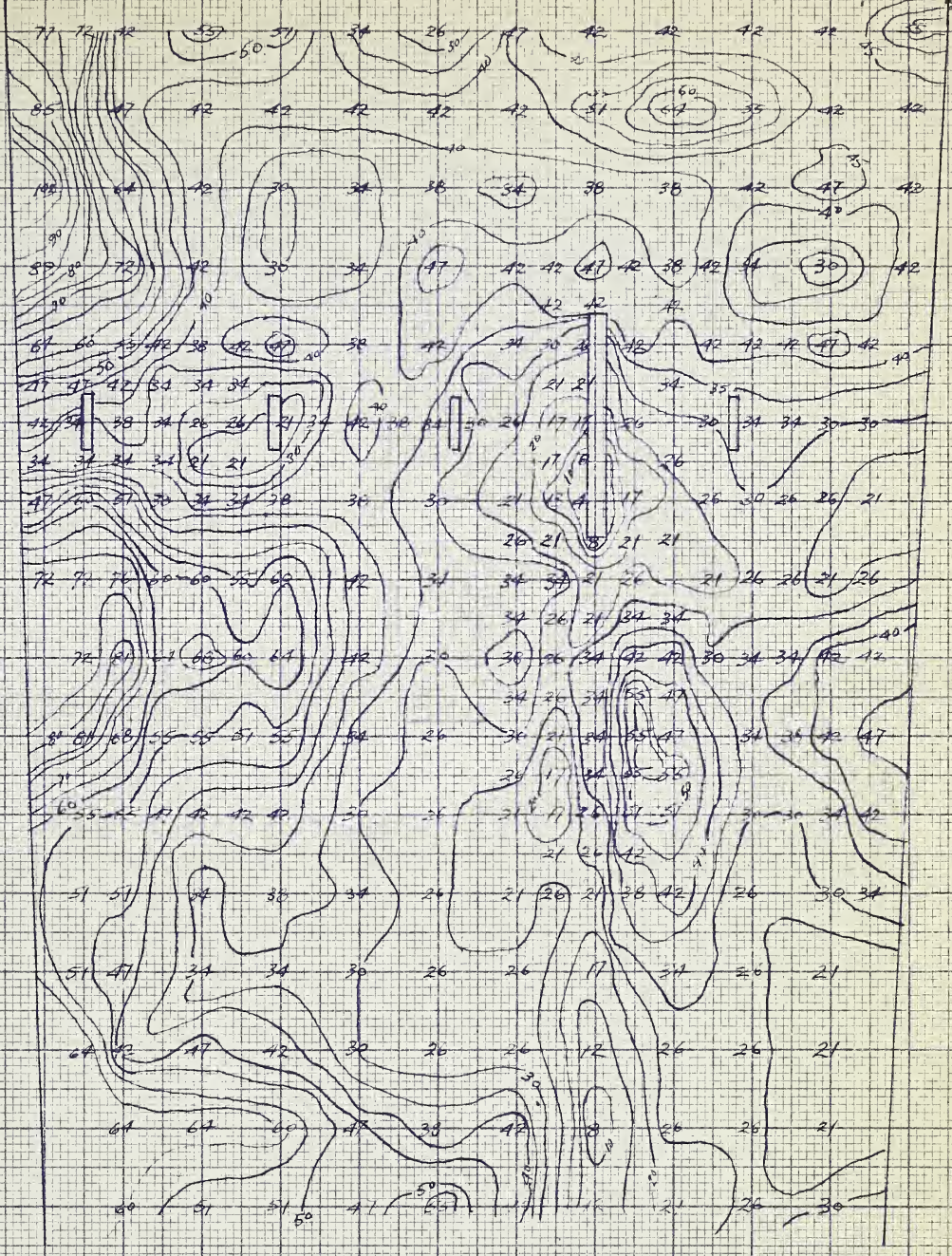


fig (a)

—— cross-section of similar prototype
x---x initial cross-section
o—o final cross-section

CROSS-SECTIONS OF MODEL PIERS
Q=0.11 cusecs. Sept/56

PLATE 7



FINAL DEPTHS AT END OF TEST. FLOW DIRECTED AT LARGEST PIER AREA.

CONTOUR INTERVAL 5'
Q = 0.11 CUSECS 1954

PLATE 8

PART III

A - PIPELINE CROSSING

B - TUNNEL CROSSING

PART III

A - PIPELINE CROSSING

INTRODUCTION

The problem of crossing rivers for different transport media is highly important. Bridge crossings problems are gradually being overcome. Whenever a tunnel is contemplated, its construction may be far below the reach of scouring action. Pipelines are placed at great depths or suspended by overhead crossings. The procedure of excavating beyond possible scour depth or constructing a overhead crossing may be very costly. Laying the pipe two feet or three feet below average river bed level and protecting it by installing stone aprons and guide banks is an alternative that might be acceptable and cheap.

A suggested design of a pipeline crossing (Plate 10) with stone apron protection at an actual site (Plate 9) in a mobile bed river was given by Professor Blench. The pipeline was in an area where the river was deep at the outside of a bend with a large shoal on the inside. A guide bank was designed to be built on this large shoal and act as a bank of the river. This guide bank would prevent the river from depositing a shoal in the deep region and eroding the present one during the course of its river changes.

The pipeline, guide bank, and opposite bank

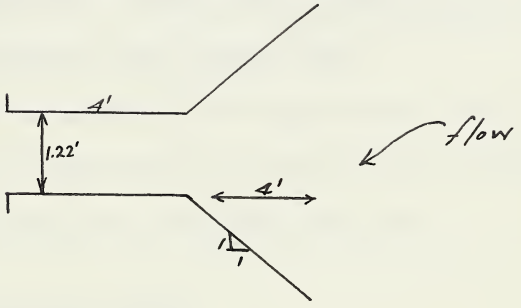
were protected by stone. Behaviour of such stone aprons has been studied in two dimensions (1) while three dimensional behaviour under the conditions presented by a variety of conventional river works does not appear to have received much attention. It is hoped that the following results assist in furthering apron studies.

EXPERIMENTAL PROCEDURE

Attainment of steady channel conditions.

Since the river tray had been used in analysis of another problem before, the existing conditions such as meander pattern dependent upon discharge, and two small guide banks 1.22 feet apart - 1.5 feet in length, were utilized in the pipeline crossing problem. One thing was changed and that was to lengthen the non-erodible guide banks. They were increased to approximately four feet in length. Two diverging non-erodible banks extending from the upstream end of our model channel were installed for a distance of four feet at a 1 to 1 slope (fig.b). These banks for the model were of ¼ inch plywood.)

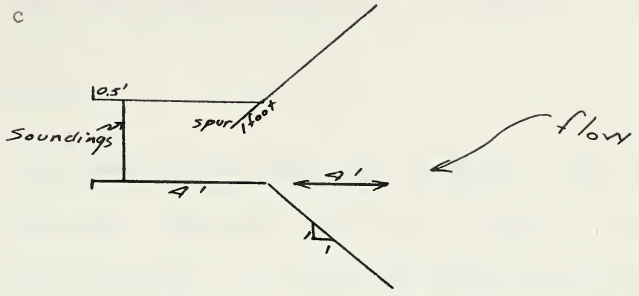
Fig. b.



The limitations of the tray's weir imposed a maximum discharge of 0.11 cusecs. This discharge of 0.11 cusecs was continued for 140 hours. During the interval of time, elevations from water surface to model bed were determined and recorded. These soundings (fig. c without spur) were taken every 0.2 feet and the reoccurrence of an average depth of 0.17 feet in the soundings indicated that

this value could be the regime depth.

Fig. c



The next step was to obtain a deepening of the regime channel on one side to simulate the prototype. This was done by placing an obstruction in line with one of the diverging sides and extending one foot into the channel proper. (fig. c) Soundings were taken at 0.5 feet along the 4 foot portion and at 0.4 feet across the channel. Records of the scour along the jutting obstruction like a spur were analyzed. In this case the scour hole at the nose was approximately twice regime depth.

After an interval of 97 hours (total time 237 hours) a general cross-sectional profile fairly representative of the prototype was obtained, (Plate 9).

Determination of Scales.

The width of the model of 1.22 feet corresponding to the width of 650 feet at low water level elevation 83 in the prototype (Plate 9) was imposed. The horizontal scale is then 1.22 feet equal to 650 feet or 1 foot equals 533 feet.

The value of 0.17 feet for regime depth represents 40 feet of depth in the prototype. The vertical scale is

0.17 feet equal to 40 feet or 1 foot equals 235 feet.

The scale ratio of vertical to horizontal is 0.44.

Discharge.

The object of the model was to observe scour under extreme conditions. Therefore there was no need to run anything but high discharge. As mentioned before the maximum discharge attainable was 0.11 cusecs.

Model Stone Size.

The stone protection was assumed to scale according to the vertical (or depth scale) so that eighth inch material selected would represent 30 inch rocks. The specific gravity of the stones used was 2.5. Therefore these 1/8" stones (30 inch) model would be of about 1 ton in weight scaled to prototype size.

Model Layout.

The plywood banks were removed and replaced by the sand of the river tray. (grain size 0.27 mm). A small piece of galvanized pipe ½ inch diameter was placed across the river at elevation 65 and sloping upwards towards the guide bank where the elevation was 70. The stone apron and guide bank were then constructed according to the design on Plates 10 and 11.

RESULTS

A small flow was started with the stone apron and guide bank receiving the full flood flow of 0.11 cusecs. in approximately 20 minutes. In another 30 minutes the stone apron was covered by a large sand wave. The origin of this wave could be attributed to a wave just beginning to enter the vicinity when the flow was stopped for model construction or the substituted sand banks were unable to withstand the erosion, Since no progress would be attainable for a long period, with the obstruction, the flow was discontinued and a large channel was excavated upstream to trap sediment and let sediment free water erode the transplanted sand bar. A number of flows were used for attack on the stone apron. These flows were 0.06 cusecs for 46 hours, and 0.11 cusecs for 64 hours. The smaller flow was used to gradually adjust the channel for larger flows.

The stone apron launched as expected with the dimensions and appearance shown on Plate 12. During this interval, the nose of the guide bank was protected by the sand bar and no attack on the guide bank occurred. This effect on the model could very well indicate that no attack would occur on the guide bank (prototype) while the river was flowing in its present course parallel to the guide bank.

To observe whether the guide bank had been adequately designed in the prototype, the flow in the model was directed at the upstream end of the bank. The sand

bar in front of the guide bank was removed in 238 hours with the stone launching to elevations 55 and 60 in another 54 hours. (Plate 13)

SUMMARY

From Plate 12 when the flow was in a direction parallel to the guide bank as is now present in the prototype, scouring upstream of the apron occurred and the launched rock displaced the hole further upstream away from the pipeline. In the articles on such happenings the launching may take place at a 1:2 slope. In this case the final slope was approximately 1:7. Why there was no scour downstream of the stone apron protecting the pipe, will be explained in a section describing a stone protected tunnel crossing.

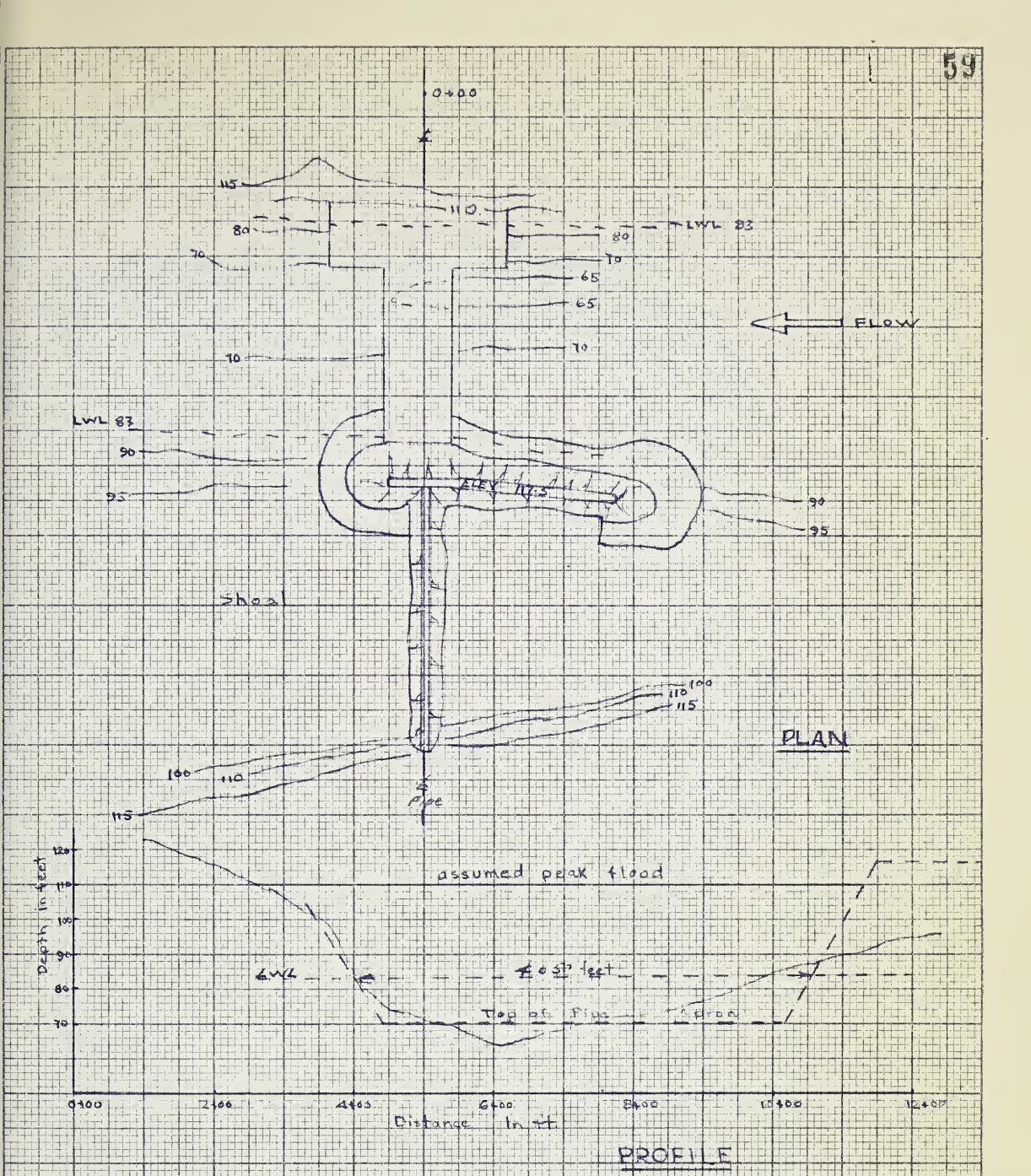
The flow was now directed against the guide bank, about 30° to the centre line of the guide bank. The original hole was filled in and a long scour hole at the base of the bank was obtained. The depths were not excessive compared to some prototype scours discussed. Plate 13 shows the appearance of the scour hole obtained. The stone launched at a slope of 1 upon 5 instead of the theoretical 1:2. At the junction of the guide bank stone protection and the pipeline stone protection, no indication was found whether more stone launched from one part or the other or that excessive action caused deeper scour. Probably launchings took place evenly.

Another model was constructed with the same dimensions and stone protection in order to obtain the same reactions as before. The flow directed against the guide bank was at about 30° to the guide bank. Photo 12 shows the

initial appearance and is of the same dimensions and stone protection as on Plate 10. For this particular run the directed flow was very severe and only a half hour was needed to notice the stone launching from guide banks and the crossing apron. Within two hours complete destruction occurred, which may be due to the breaching of the dike and the quickening action of the sand that was used in the construction of the guide bank. This phenomena is shown on Photo 13. Essentially the launched stone has taken shape as in the preceding model, with the stone launching to a cone shape from a circular nose construction.

The stone has launched in a tapering form of protection, that is, 2 layers close to the top and one larger at the bottom, while at the intersection, there is no evidence of one part launching excessively compared to the other. In both cases, the final launching is very close to the pipeline. Now the question is, is it the fault of the design where more stone was determined for launching downstream, and did not occur, or the model layout of the stone apron is dependent upon certain model scales. It can be shown that excessive scour will occur upstream of a stone apron crossing and therefore equivalent amounts should be designed on both sides of the pipeline or more stone on the upstream side. The model stone aprons were laid to the horizontal scale. One would say that if the stone launches to a depth scale why not lay the stone to the vertical scale. This factor should be reviewed very closely. Depending upon

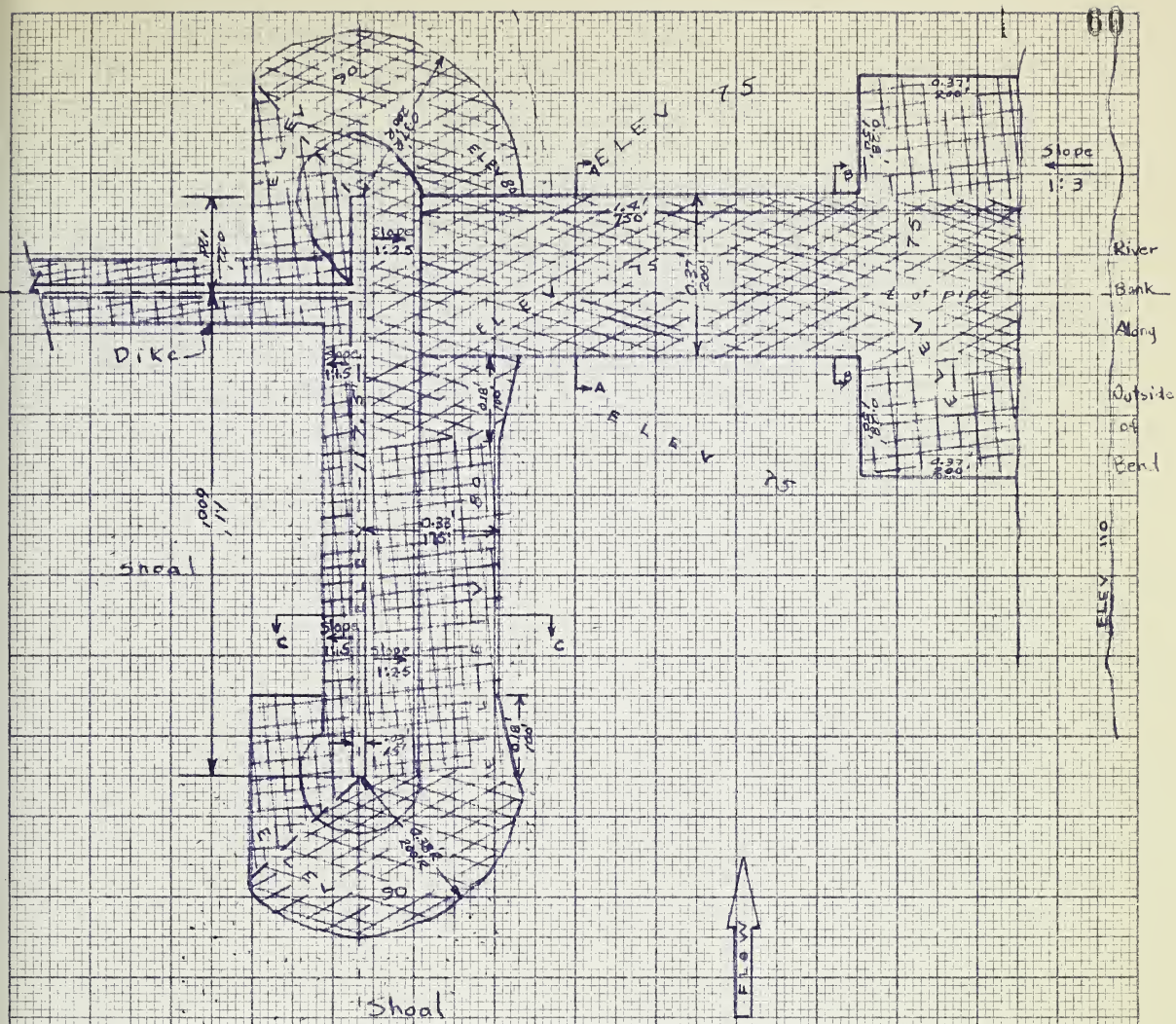
the scale ratio, laying out of the stone apron to a vertical scale would practically cover the entire channel and one would obtain a type of stone rapid design (see Photo 14). The ratio of vertical to horizontal should be approximately less than one and the stone protection area measured to the horizontal scale, but with the volume of stone needed for it to launch to vertical scale.



SKETCH OF

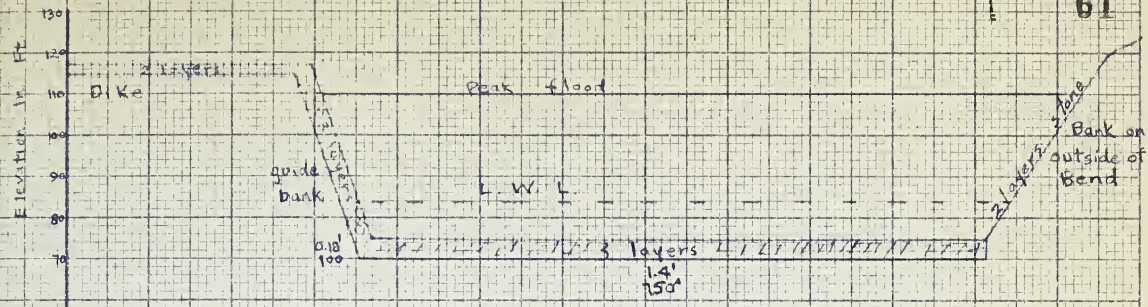
Plan and Profile of Pipeline Crossing Protected by Guide Bank and Stone Aprons (Prototype)

PLATE 9



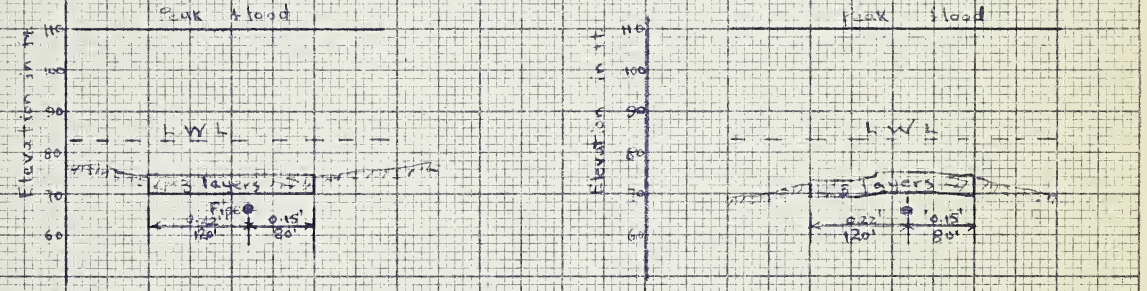
DESIGN OF STONE APRON PROTECTION AND GUIDE BANK

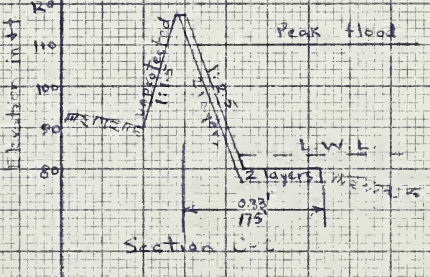
PLATE 10



Section along Pipe Centreline

Section along Guide Bank Centreline

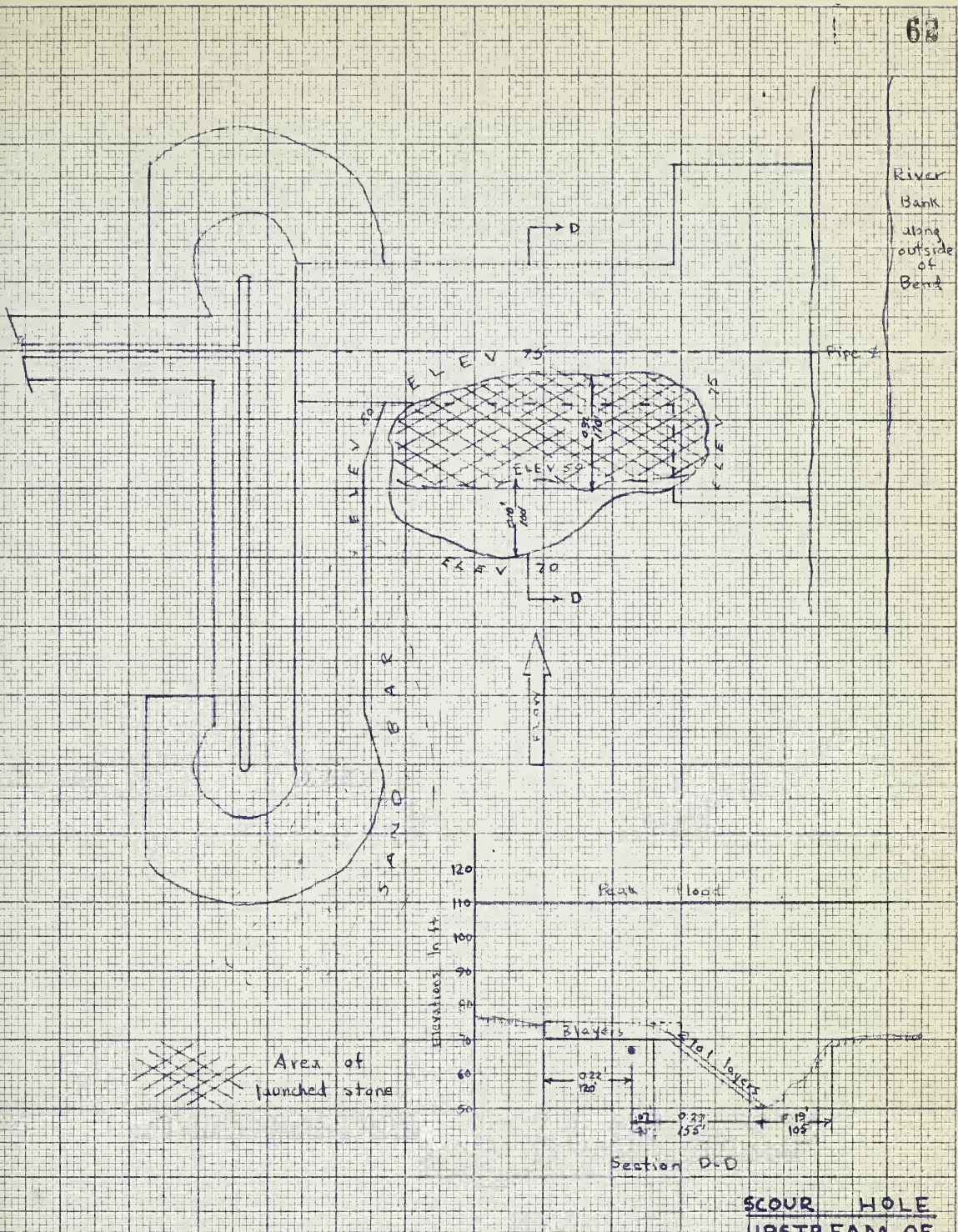




DESIGN OF STONE APRON PROTECTION AND GUIDE BANK

scale 1" = 200'

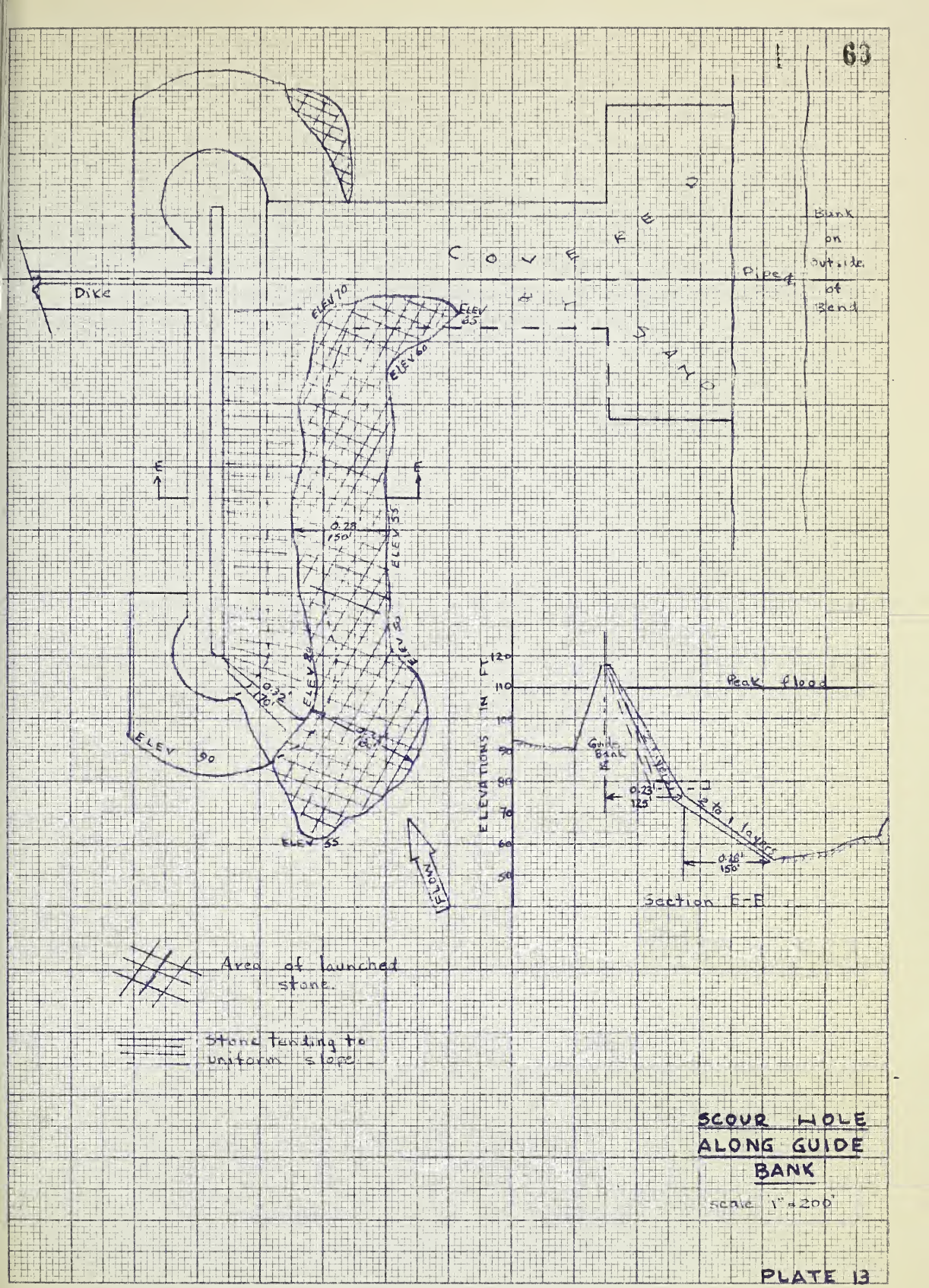
PLATE II



SCOUR HOLE UPSTREAM OF APRON CROSSING

scale 1" = 200'

PLATE 12



SCOUR HOLE ALONG GUIDE BANK

scale 1" = 200'

PLATE 13

PART III

B - TUNNEL CROSSING

INTRODUCTION

Further studies on apron launchings were made available by a proposed design of a stone protected concrete tunnel crossing. The tunnel was to be located in a navigable channel of fairly uniform cross-section. A model was made representing a section in the deepest part of the prototype channel. This model portion was then placed in the glass walled flume and studies of stone launching conducted. Results from this model indicated that a water surface drawdown will occur over the weir-like obstruction, the weight of stone to be used in the prototype would be inadequate in the navigational channel, and the stone designed for slope protection would launch satisfactorily.

EXPERIMENTAL PROCEDURE

Attainment of steady conditions.

The flow through the flume was continued until there was no change in the mean depth of flow and whole bed consisted of waves. The mean grain size of the bed material was 0.27 mm.

Discharge.

The initial discharge used was 0.27 cusecs and then 0.35 cusecs. The measurement of the discharge was by a sharp crested weir which was calibrated by assuming the discharge through a rotameter assembly as being correct. (See Fig. 5 for calibration curve).

Determination of Scales.

The vertical scale was determined by the mean model depth of 0.31 feet for 0.27 cusecs. representing 55 feet in the prototype. Therefore the scale is 1 foot is equal to 178 feet.

The cross-section dimensions of the tunnel were in proportion to the depth and since a unit length of section was removed for model construction, the horizontal scale chosen is the same as the vertical. Furthermore the flow pattern must be the same in both model and prototype. This can only be achieved if dynamical similarity exists.

Model Layout and Dimensions.

Plate 14 shows the prototype and model dimensions used for the tunnel crossing study.

Four layers of stone were placed in order that the final protection would be in three layers.

Apron Stone Size.

1/8 inch chips of specific gravity 3 were used. Scaling by depth scale to the prototype they would represent 1.8 foot size rock of weight 1100 pounds. 3/8 inch size chips were also placed for study, this size would represent approximately 6 foot rock of weight 20 tons.

Procedure.

Continuous flow over the model was made. Observations of how the stone launched were kept. Obstructions were placed close to the roof and their effects recorded. Since only a ship could produce the turbulence to move the stone, the model depth had to be raised in order for it to correspond to a section of tunnel in the navigational portion. This additional depth of 6 feet was obtained by increasing the discharge to 0.35 cusecs. At the same time the larger size of stone was placed on the roof for study under effects of a ship's propellor and structure. A model boat was used to produce the prototype ship effects. The downstream portions of rock were continously covered by the

transported sand and no launching could occur. Therefore for one run, no stone was placed on the downstream side and for two runs stone at different slopes was tried.

RESULTS

Run 1 -- 0.27 cusecs. - both sides and roof of tunnel protected by 1/8" stone.

The initial appearance is shown by Photo 15. A heavy black line on either side of the model represents the mean bed level from continuous flow to obtain steady conditions. The model was under scouring attack for 49 hours and Photo 16 shows the final appearance for this interval of time.

The downstream portion has not launched at all. It has been covered by the sand that has been carried back by a returning jet. This division of sand movement occurs approximately at the location of the arrows on the Photo 16. The bed has been scoured below mean bed level. Photo 16 does not show the bed sloping upwards again to the previous level where bed waves are again dominant.

There was a definite drawdown of water surface over the tunnel section. The value was 0.03 feet model or 5 feet prototype.

Wherever an obstruction was placed in the flow approximately 5 to 8 feet above the tunnel roof a movement of the stone on the roof occurred. On Photo 16 this rock can be seen downstream. The unstable portions seem to be the stone on the roof and stone protection on the 1:5 slope at the edges of the tunnel.

Run 2 -- 0.35 cusecs.

The depth of flow was increased another 6 feet to represent a section in the navigational channel whereas the previous section was outside the navigational channel. The model was under attack for 51 hours. During this time the model boat was placed over the roof and movement of stone occurred.

Run 3 -- 0.35 cusecs - top of the roof protected by 3/8" rock (prototype 6 foot - 20 tons).

During this interval of 18½ hours the model boat with its rotating propellor moved the larger size stone. The propellor actually touched some portions and this probably started the first few moving and the rest moved much more quickly over the exposed tunnel roof which was very slippery.

Run 4 -- 0.35 cusecs. - downstream slope replaced by 1/8" stone at 1:2 slope.

The upstream stone protection was untouched, the roof protection was left alone with the same stone size of 3/8" as in Run 3. Since the downstream protected slope had been covered by sand, the rock and sand were removed to be replaced by new protection at a slope of 1 upon 2 beginning at the tunnel edge. This new slope is the slope that the rock is to launch to. Photo 17 shows the initial appearance. After 98 hours of 0.35 cusecs flow the downstream slope is again covered by sand (Photo 18).

Run 5 -- 0.35 cusecs - downstream slope just sand backfilled at 1.2 slope.

The upstream and roof protection were untouched. On the downstream face the rock was removed and only sand backfill at 1:2 was left to the elements of scour. The downstream slope has the same appearance as Photo 17 except the stone is removed. No scour on the slope developed after 48 hours of attack (see Photo 19). A spur was placed upstream to cause a different flow pattern over the tunnel. No drastic scour holes occurred on the downstream portion and the upstream stone protection did not fail.

SUMMARY

The model results indicate that stone of approximately 20 tons in weight would have to be placed on the roof within the navigational channel to withstand the turbulence caused by ships' propellors. The stone protection on both sides of the tunnel is adequate. On the upstream portion launching of stone will occur to a maximum depth measured to the trough of a bed wave. Cross-currents may actually increase the depth but this cannot be foreseen with the type of testing procedure used. Downstream stone protection can be decreased to the 1:2 slope design, since no launching of this portion will occur, any protection placed in this area will be merely a safeguard against turbulence from ship propellors.

The decrease in water surface elevation over the tunnel is very important. The drawdown is about 6 feet, hence a 42 foot depth of water is decreased to 36 feet. If any large ships of draft 26 feet to 30 feet must travel upstream, the stone will be displaced easily.

CONCLUSIONS - for Part III - A & B

The model of a pipeline crossing protected by stone indicated that a prototype construction could be feasible. A number of precautions indicated by the model would have to be taken:

1. Where two layers of rock were required on the river side of the guide bank, three layers should be specified.

2. The upstream stone portion protecting the pipeline should be increased in order that the width of stone aprons for launching on either side of the pipe be equal or in a ratio of upstream to downstream protection not less than 1.

In further studies of similar models layout of stone apron areas should be to a scale that will result in the model appearing in proportion.

The study made in Part III - B - Tunnel Crossing - showed that downstream scour from flat apron protection occurs some distance away, while the sections close to the part being protected may be covered by sand. This phenomena was observed in Part III - B - Tunnel Crossing, a two dimensional study and also in Part III - A - Pipeline Crossing - where the three dimensional effects were taking place. The discharges in all cases represented peak values. If the models were subjected to a fluctuating discharge different results could be obtained. When another similar model is made the author suggests a varying discharge be used. These

stone aprons may force low discharges into waterfalls at their edges where scour will be severe.

From these studies the coefficients of scour that can be expected upstream of a stone apron crossing is 1.5. This value indicates that scour will occur to a depth 1.5 times the mean depth of flow for the peak discharge in that area measured from water surface. The coefficient along a guide bank obtained from Part 3 - A is 1.5. Therefore the design of stone aprons and guide bank protection can be based on these coefficients if the attack is not too severe. These coefficients will have to be increased if the guide bank is very short, say its length would be less than 3/4 of the distance between guide banks.

Inspection of the launched aprons indicated that it was a very rare occurrence to obtain protection on the launched slope of more than one layer of rock. This factor itself may indicate that less stone should be designed to be placed for launching. From the models there was no indication of excessive amounts of rock left for launching. The reason may be that some of the lighter rock has been carried away by the flow and this deficiency probably balances the overdesign.

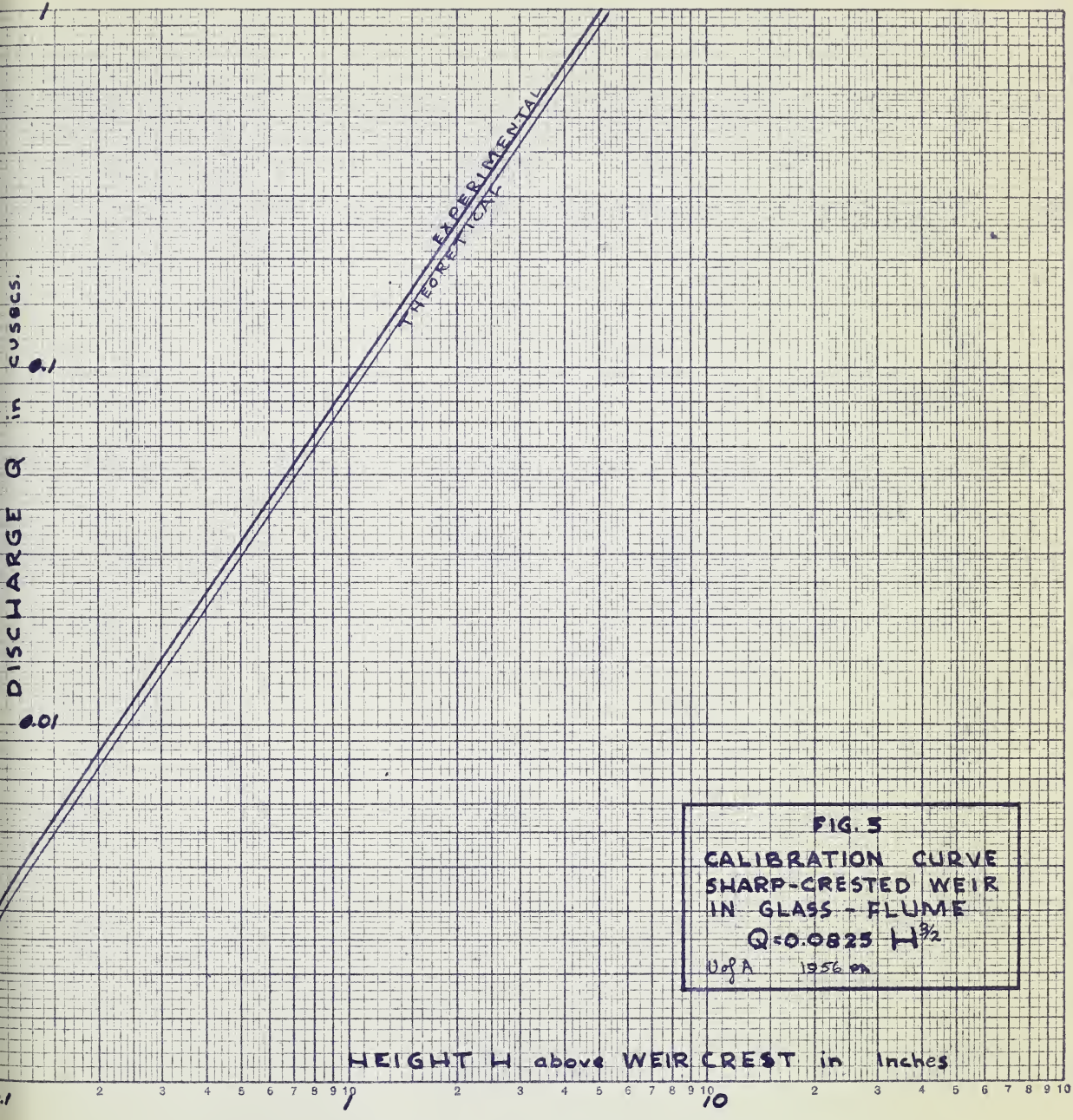
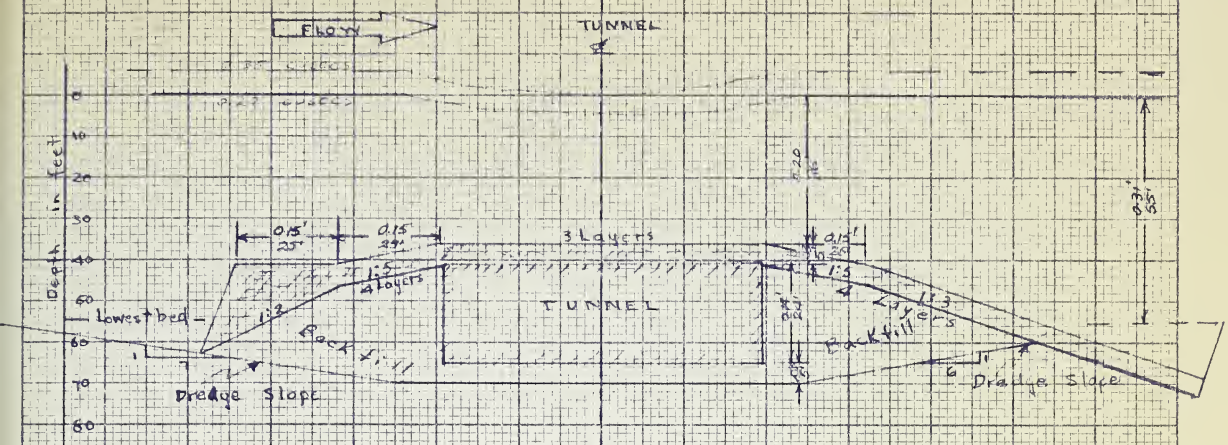


FIG. 5
CALIBRATION CURVE
SHARP-CRESTED WEIR
IN GLASS - FLUME
$Q = 0.0825\ H^{3/2}$
U of A 1956 m



MODEL CROSS-SECTION
scale 1 inch = 40'

TUNNEL CROSSING WITH STONE PROTECTION

PLATE 14

P A R T IV

MODEL DISTORTION

PART IV

MODEL DISTORTION

INTRODUCTION

Three identical submarine crossings were tried out as a model test in the study of stone aprons. They were set parallel to each other on the centre line of the tray, inclined to it at meander cross-over angle and located at a distance half a meander length from each other. In this way two of the crossings would have very disadvantageous approach conditions.

The construction of the model system was based on a number of questions that were to be answered for an exercise. Prophesies by the theoretical answers were to be checked by model results.

The attack by the flow in this model was very severe, aprons were washed out and scour holes extending across the width of the model submarine crossings developed. The original model channels were filled in by sand and new channels eroded on the opposite sides. A record of this model distortion was obtained on 800 feet of 16 mm film.

EXPERIMENTAL PROCEDURE

The fixed conditions upon which the calculations were based, for model design, were discharge, bed factor, and depth(which is partially dependent upon discharge). Since the test was in the interest of aprons, a fluctuating discharge was not required. The maximum attainable discharge was 0.11 cusecs. A depth of approximately 0.2 feet would result from the fixed discharge in the east river tray. This fine sand of 0.27 mm median grain size fixed the bed factor at a value of 1.1.

Slope

Assuming a side factor of 0.15, the regime slope for 0.11 cusecs. is 0.06%.

A check for laying out the meandering channel by the calculated slope is, the slope across a meander bend is 3 to 4 times the calculated regime slope. These values are based upon the assumption that the length around a bend is 1.5 to 2 times the distance straight across the bend and the friction factor around the curvature of the bend is about two. Therefore a slope 3 to 4 times regime slope straight down the tray is comparable to the slope measured along the river channel.

Meandering Channel

The channel was constructed in a meandering condition based upon the formula meander length being approximately

equal to 36 $Q^{1/2}$, where Q is measured in a canal section for steady discharge. The meander length was computed to be 12 feet, from this value the meander breadth was 6 feet, assuming meander breadth was one half the meander length.

Guide Banks

The distance between guide banks is 1.17 feet. This value was obtained from the equation Q=V b d, where Q is the model discharge of 0.11 cusecs., V the velocity determined from V^2/d = Fb (bed factor 1.1), b the width between guide banks, and d the depth of flow 0.2 feet.

Three types of guide banks were made, the first one had vertical ends (Apron crossing 10) and the other two had sloping ends. One with 1:1 slope(Apron crossing 20) and the other with ends at 1:2 slope (Apron Crossing 30).

Stone Aprons

The stone was the same 1/8" size as used in other models of 0.11 cusecs. discharge, and approximately 0.2 feet depth of flow.

The assumed scour upstream of the stone apron is 0.2 feet below mean bed level (a coefficient of 2) and scour downstream of 0.15 feet below bed level (coefficient of 1.5). The rock should launch into a final position of 1:2 slope. Therefore the rough formula for laying the stone on the initial flat level of 1.5 times the scour depth below bed level will be used. Then total width of apron at mean bed level before scour is 0.52 feet. This value was obtained by addition of flat portions required upstream(1.5 x 0.2 = 0.3)

and downstream (1.5 x 0.15 = 0.22). The stone will launch to a section as shown

. A level portion should be retained for further protection as shown in the following diagram

. Hence an arbitrary 0.08 will be added to 0.52 feet to obtain a final design width for the apron of 0.60 feet.

Model Layout

Photo 20 shows the appearance and location of the test sections. The aprons are 0.6 feet wide by 1.17 feet long. The length 1.17 feet being the distance between guide banks. The submarine crossings are spaced 6 feet apart and practically block the channel.

Procedure

The bottom part of the tray and channel was filled with water in order that the oncoming flow would not scour the aprons and erode the sides too badly. Individual photographs and 800 feet of film were made of the progressive changes that occurred in 165 hours that the models were tested. Two undergraduates recorded the depth of scour at the guide banks and also plotted scour depth vs time (7).

RESULTS

After 6 hours

Photo 21 shows the two imposed channels between the aprons being filled with sand. Sand has been scoured out in the bend above 10 to be deposited in the bend above 20. A straight channel is being eroded from 10 to 20 and the beginning of the same action is occurring between 20 and 30.

Apron 10

Photo 25 shows the apron and guide bank at 0 hours. The right guide bank looking upstream is practically blocking the channel. This guide bank with the vertical ends and its location, will result in producing very disadvantageous approach conditions.

Photo 26 indicates the stone apron has received a very severe beating. The flow has completely removed the apron on the left side, depositing the stone approximately three feet downstream. This is due to the right angle turn the river had to make to enter the guide bank channel. A hole has been scoured to such a size that the effects are felt on the opposite bank. This is a very good case of model distortion.

Apron 20

Initial appearance is given by Photo 30. The opposition to flow is not too great, due to the sloping ends.

Flow pattern over these ends is not diverted too

quickly to create excessive turbulence for increased scour.

Photo 31 shows the main flow is occurring parallel to the guide bank walls. This is due to the direction being imposed on the flow by Apron 10. The stone has launched and portions of the rock have been carried downstream. The scour holes at the upstream noses of the guide banks are smaller and are not influencing the opposite bank or each other. The channel downstream has not been straightened to the extent as crossing 10 straightened the flow. Thus the sloping ends are a factor in forming less obstruction to the flow, which is clearly indicated by the two submarine crossings with such ends.

Apron 30

The initial conditions as obstructions to flow through the area are not severe. Photo 35 is the initial appearance. Photo 36, indicates that the attack was much less drastic. Part of the apron has launched and other sections have been covered by sand. The length of the scour hole has touched the apron to partially influence the launching.

After 60 hours

Photo 22 shows the channel between the outermost aprons has been straightened and the flow is gradually being switched to the opposite side. Portions of the original channels have been left open, a condition that occurs in the prototype.

Apron 10

Further erosion has occurred on the outside of the bend (Photo 27) and at the exit of the Apron the original bank has been removed. The scour holes at the upstream noses have merged, while scour holes at downstream noses have not been obtained.

Apron 20

Photo 32-by the appearance of the bed waves, the flow is directly upon the guide bank. The bed elevations have increased considerably and large amounts of sand are being transported. At the exit there is a slight turn of the flow to follow the original bend. The stone apron is still intact and covered by sand. The shapes of the scour holes do not seem to be well defined for sloping end guide banks compared to guide banks with abrupt ends.

Apron 30

Photo 37 shows the guide banks very nearly covered by sand, especially the downstream nose. A large wave of sand is intruding the area between guide banks, but no sand has been deposited along the guide bank. The apron is still probably intact under the coverage of sand. The slight turn of the flow from the exit of Apron 20 is now directly in line with Apron 30. At the exit of Apron 30 it has been diverted to the right again looking upstream and further erosion of the bank that was obstructing the exit has occurred.

Again the photograph shows the old channels are still open.

After 120 hours

Photo 23,- after the bank in front of Apron 10 was eroded, the channel width at the top of the tray has become wider. This is shown by the small upright sticks which were placed at the meander breadths and on the outside of the bends. Further erosion has occurred to develop the new channels on the opposite sides. At the exit of Apron 30 a large increase in elevation has been obtained. This may be due to the enormous amount of sand eroded from the banks to form the new channels.

Apron 10

The bank in front of the apron has been removed and a new attack on this area has developed. The old channel to the right is just filled with water and no erosion is taking place. The flow is now attacking from a reasonable direction. The attacks on the right bank past the exit of Apron 10 is less severe.

Apron 20

The attack is gradually developing from the right as the right bank is being eroded (Photo 33). Upon exit from Apron 20 it swings to the left for further widening of the channel before entering Apron 30.

Apron 30

The flow has curved again to enter Apron 30. From Photo 38 the large scour hole developed from the left guide bank encroachs upon the right guide bank to extend the jetting action beyond the guide bank area. Considerable erosion on the right sand bank has occurred. The bed has risen to such an extent at this particular apron that the water is nearly flowing over the wooden guide banks. The approximate increase in elevation is 0.2 feet.

After 165 hours

The rate of bank erosion has slowed down considerably as shown in Photo 24. The new channels are being deepened by scouring and seem to be approximately the same depth as the original channels. They are directly opposite to the old ones. Definite banks were confining the flow originally, but now water is flowing through a very much increased width expecially between the guide bank locations.

Apron 10

The attack is definitely from the left (Photo 29) with the old channel being a spill area. The flow still has to make a right angle turn to develop scour holes that are typical at vertical end spurs or guide banks.

Apron 20

Photo 34 - the attack is from the right where

originally it was from the left. The scour hole extends completely across the two guide banks. Bed elevations have increased and the flow of water, being disturbed less by sloping ends than vertical ends. This is clearly shown by the bed waves which are moving in the same direction above and below the apron crossing.

Apron 30

Photo 39 - the water is flowing over the dikes on the left side looking upstream. The same effects that occurred at Apron 20 are found in this area. Main channel of flow is on the opposite side, with the old channel above Apron 30 still open but at the exit it is practically filled in. The size of the scour hole is larger than at Apron 20 but this is due to increased portion of guide bank slope being exposed to the flow.(ends slope 1:2).

SUMMARY

The question of how much scour to expect at obstructions cannot be answered too well by the results from this model. Most models scale their depth of scour to the prototype dimensions very well. The slopes of the sides of the scour hole are the same in model and prototype. Thus where the scour edge extends practically half ways along the guide bank in the model, it is not at all representative of what occurs in the prototype. In the prototype the edge of the hole is very close to the nose of the bank. Good examples of such distortions are seen on Photos 21, 26, 27 and 40. Photo 40 has a very large hole as developed by the direct jet impinging the non-erodible wall. Compared to the width of flow, such a scour in the prototype would not occur. The depth of this hole is very representative of the prototype when scaled correctly.

The scour measurements upstream and downstream of a stone apron crossing could not be obtained. The apron stone definitely launched but the distortion developed by the scour holes would not give reliable answers to scour measurements. Studies made in the previous section will be of assistance for this problem.

The depths to expect at guide bank noses and along their length may be obtained from Bibliography 7. Their data will give values for final depths. The plots of scour depth vs time resulted in curves that showed scour increasing

with time very quickly and then leveling off, but still increasing slowly. Other plots, Bibliography 3, Ahmad, show the same results. Therefore final measurements of scour depths should be taken approximately after 180 minutes have elapsed.

Some values that have not been too greatly affected by the model distortion were chosen to assist in verifying some coefficients that are being used. For instance the right guide bank of Apron 10 looking upstream (Photo 26) has a large scour hole. The attack in this case has been very severe and the coefficient obtained is 2.35.

The sloping ends on guide banks have a very definite effect on scour holes. The holes are much shallower and elongated. A coefficient will be quoted but further work is necessary and this value of 1.5 is subject to revision. If vertical ends had been used on all the guide banks, coefficients could have been checked. Finally, the straightening effect, would have been developed at a quicker rate with vertical ends and a straight canal between submarine crossings may have been obtained.

The terrific increases in elevation merit discussion. A slope for the channel was calculated assuming no charge being present in the flow. This factor may be important but to determine a charge to use in the revised slope equation would have been difficult. The slope layouts were made from the remaining pipeline apron crossing which was used as a controlling point. Setting the slope from

the pipeline crossing may have established too steep a gradient and erosion had to occur to flatten the slope. The actual slope should have been measured from the exit weir at the bottom of the tray and the old pipeline crossing removed.

CONCLUSIONS

The preceding model analysis gave important information on bed waves, model distortion, effects of sloping ends on protection works and types of scour holes. Further studies should be made on guide banks as modeled by Aprons 20 and 30. If a sloping guide bank was used in place of Apron 10 the results may have been quite different, but only additional model runs will clear the mystery. Definitely sloping guide bank ends reduce scour depths and scour effects, as indicated by the model experiment.

An alternative to pitched islands for river training could be the construction of such guide banks as were modelled. The expense will probably be formidable, but the river may be straightened.

Further work on a model similar to the one tried will give valuable answers. Changes could be made in the width (to be increased) between guide banks, an increase in discharge and better control at the start of the model in order to prevent movement of the stone protection.

Close study should be made ~~to prevent~~ of the causes of aprons becoming covered by the sand. This phenomenon has occurred in practically all apron studies made by the author.

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APPENDIX A

TABLE 8-I

SHOWING MAXIMUM DEPTH OF SCOUR IN RIVERS AT BRIDGE PIERS, GUIDE BANKS, GROYNES, SHARP BENDS AND DOWNSTREAM OF BRIDGES

No.	Site.	Q (maximum) (cusecs).	d_s = max. depth of scour (ft.).	Year of observation.	m = mean grade of bed material (in mm.)	$fd = 1{\cdot}76\sqrt{m}$ assumed as an approximation.	$d_s \times f_d^{\frac{1}{3}}$	$D_{Lacey} = {\cdot}473\,(Q/fd)^{\frac{1}{3}}$.	d_s/D_{Lacey}.	References.
			8-I. SCOUR ROUND PIERS OF BRIDGES.							
1 (a)	The Ganges near Hardinge Bridge (Bengal and Assam Railway).	16,75,000	117	1938	0·37	1·07	119·7	54·93	2·13	A. R. (T) 1937-38. p. 66; 1938-39, p. 22 and 1939-40. p. 23; Bombay Tech. Papers 55 and 58.
2	The Brahmaputra at Amingaon ...	22,50,000	104	1938	0·39	1·10	107·4	60·09	1·73	A. R. (T) 1938-39, p. 23.
3	Meghna Railway Bridge (B. & A. Railway) ...	8,00,000	94	1938	0·32	1·0	94	43·91	2·14	Appendix 1 of A. R. (T) 1944 (pp. 78-81)
4	Karnaphuli Railway Bridge (B. & A. Railway).	3,09,000	57	1938	0·32	1·0	57	31·99	1·78	Do. do.
5	The Jumna at Delhi Railway Bridge (E. I. Railway).	43,000	37·5	1941	0·32	1·0	37·5	16·58	2·26	A. R. (T) 1937-38, p. 65.
6	Gandak Railway Bridge at Bagaha (O. & T. Railway).	3,00,000	74	1924	0·38	1·08	75·9	30·87	2·40	A. R. (T) 1944.
7	Par Railway Bridge (B. B. & C. I. Railway) ..	1,68,000	58·5	1941	0·33 (assumed)	1·0	58·5	21·1	2·24	Appendix 1 of A. R. (T) 1944 (pp. 78-81).
8	The Indus in Rohri-Bukkur Gorge ...	4,14,000 (2/3 of 6,20,000 cs., 1/3 flowing through Bukkur–Sakkur Gorge).	82·7	1933	0·17	·71	73·8	31·46	2·35	See Figs. 36 and 37, A. R. (T) 1937-38.

TABLE 8-I *contd*

SHOWING MAXIMUM DEPTH OF SCOUR IN RIVERS AT BRIDGE PIERS, GUIDE BANKS, GROYNES, SHARP BENDS AND DOWNSTREAM OF BRIDGES

No	Site	Q (max. value) (cusecs)	d_s = max. depth of scour (ft.)	Year of observation	n = mean grade of soil (value of $\frac{1}{m}$ in Manning's formula)	$d_f = 1.76\ q_m$ assumed as an approximation	$d_s = f\frac{b}{d}$	$D_{Lacey} = 4.75\,(Q/f)^{\frac{1}{3}}$	d_s/D_{Lacey}	References
1	[illegible]	[illegible]	45	August 1938	0·34	1·01	25·24	14·40	1·73	
2	[illegible]	[illegible]	48	June 1938	0·32	1·0	48·0	18·54	2·59	
3	[illegible]	3,00,000	41·7	July 1939	0·37	1·07	42·65	21·68	1·93	
4	[illegible]	1,00,000	41·4	June 1936	0·39	·96	41·97	22·97	1·80	
5	[illegible]	1,50,000	45	August 1938	0·34	1·05	45·44	25·21	1·78	
6	[illegible]	1,65,000	65·2	July 1933	0·24	1·03	65·85	25·81	2·53	
7	[illegible]	93,787	46·3	August 1942	0·24	·80	38·52	20·47	1·98	
8	[illegible]	81,334	39·1	July 1939	0·40	1·0	39·1	18·65	2·09	
9	[illegible]	1,18,747	49·14	August 1933	0·26	·79	45·43	24·21	2·02	
									mean = 2·09 ± ·27 or ± 12·9%	

A-1

TABLE 8-II

47	Sutlej River Express Railway Bridge at Adamwahan (N.-W. Railway)	1,06,747	49·14	August 1933	0·20	79	45·43	24·2+	2·02	
									mean = 2·09 ± ·27 or ± 12·9%	

TABLE 8-II

SHOWING MAXIMUM DEPTH OF SCOUR IN RIVERS AT BRIDGE PIERS, GUIDE BANKS, GROYNES AND SHARP BENDS

No.	Site	Q max. (Cusecs)	d_s = max. depth of scour.	Year of observation.	m = mean grade of bed material (in mm.)	$f_d = 1{\cdot}76\sqrt{m}$ assumed as an approximation.	$d_s f_d^{\frac{1}{3}}$	$D_{Lacey} = {\cdot}473(Q/f_d)^{\frac{1}{3}}$	d_s/D_{Lacey}.	Reference
1 (a)	The deepest scour under the piers of the Railway Bridge at Hardinge Bridge	17,00,000	206	1933-35	0·39	1·10	212·6	54·69	3·77	See Bowlus Tech. Paper No. 58 A. S. C. E. for 1927 pp. 1809 to 1844 and 3059 to
1 (b)	The deepest scour near the nose of the Right Guide Bank at the Hardinge Bridge	7,00,000	159	1933	0·37	1·07	153·4	55·2	2·72	
2	[illegible]	50,000	66	1940	0·27	·92	64·18	16·96	3·9	A. R. (P) 1940-41, p. 12
3	[illegible]	10,00,000	106	1939	0·32 (assumed)	1·0	106	47·3	2·24	Appendix I of A. R. (P) 1939-40, pp. 78-81
	[illegible]	3,00,000	60		0·27	·92	58·35	32·56	1·84	A. R. (P) 1941-42, p. 29
4	[illegible]	18,000	30	1938	0·32	1·0	30	17·27	1·77	A. R. (P) 1937-38, p. 30
	[illegible]	31,000	24	1939	0·32	1·0	24	14·86	1·62	
5	[illegible]	2,00,000	65	1944	0·32 (assumed)	1·0	65	27·9	2·33	
	[illegible]	2,00,000	68	1944	0·32 (assumed)	1·0	68	27·9	2·44	
6	[illegible]	50,000	65	1935	0·32	1·0	65	17·4	3·74	Appendix I of A. R. (P) 1934-35, pp. 78-81
	[illegible]	50,000	96·4	1932	0·32	1·0	96·4	16·4	5·9	

MAXIMUM DEPTH OF SCOUR AT BRIDGE PIERS

A-2

TABLE 8–III

SHOWING MAXIMUM DEPTH OF SCOUR IN RIVERS DOWNSTREAM OF BRIDGES

No.	Site	Q max. (cusecs).	d_s = max. depth of scour.	Year of observation.	m = mean grade of bed material (in mm.)	$f_d = 1.76\sqrt{m}$ assumed as an approximation.	$d_s f_d^{\frac{1}{3}}$	$D_{Lacey} = 473(Q/f_d)^{\frac{1}{3}}$	d_s /Lacey.	Reference.
1 (d)	The Ganges downstream of bridge pier No. 3 of the Hardinge Bridge.	16,70,000	216	1938	0·37	1·07	221	54·93	3·94	A. R. (T) 1941-42, p. 31.
2	Rohri Canal downstream of Head Regulator at Sukkur.	10,900	47	.....	0·07	·47	36·55	13·49	3·48	Appendix I of A. R. (T) 1944, (pp. 78-81).
3	Rohri Canal downstream of Rohri Multan Road bridge at R. D. 52,000.	10,900	56	.....	0·056	·42	41·94	14·01	4·0	Do. do.
4	N. W. Canal downstream of the Railway Bridge near Eak at R. D. 76,000.	5,000	31·5	1942	0·30	·96	31·05	8·2	4·08	A. R. (T) 1944.
5	Rohri Canal downstream of the Railway Bridge at R. D. 75,000.	10,050	45	1938	0·32	1·0	45	10·21	4·41	A. R. (T) 1942-43, p. 21.
6	Eastern Jumna Canal downstream of Road Bridge at Malcheri at M 22·5.	2,072	31·5	August 1942	clay	.....	.....	.....	Mean = 3·98 +0·3 or +7·5%	.....
7	Eastern Jumna Canal downstream of Sarkari iron bridge at M 26·1.	2,054	24·5	August 1942	loam	.....	.....	.....	.....	.....
	SCOUR IN V. E. MODELS									
1 (e)	The Ganges near right guide bank of the Hardinge Bridge.	8·0	2·3	.....	red silt (m = 0·5mm.)	1·25	2·48	·88	2·61	Bombay P. W. D., T. P. 55.
5	The Jumna near Delhi Gate Pumping Station at spur 5.	2·8	1·0	.....	Theur sand (m = 0·24)	0·86	·95	·70	1·43	A. R. (T) 1940-41, p. 35 and 1942-43, p. 2.

A-3

No 1451 B

CENTRAL IRRIGATION
&
HYDRODYNAMIC RESEARCH STATION

(FIG 35 FROM A.R.I. 1939-40 OF C.I.&H.P. STN.)

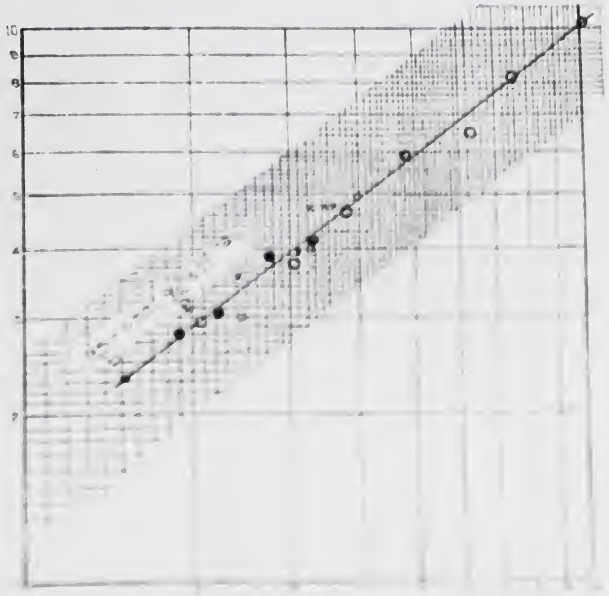
FIGURE 8-6

EXPERIMENTS WITH MODELS OF HARDINGE BRIDGE PIERS WITHOUT PROTECTION

RELATIONSHIP BETWEEN d_s = DEPTH OF SCOUR AT NOSE, b = WIDTH OF PIER AND q/c = CENTRAL DISCHARGE PER FT WIDTH $\frac{1}{5}$ OF PIER

MODEL SCALE

- × ····· 1/65 ····· } GANGES SAND MEAN
- □ ····· 1/105 ····· } DIAMETER 0.29 m.m.
- ○ ····· 1/210 ····· }
- ● ····· 1/65 ····· NALA SAND 1.3 m.m.
- ■ ····· 1/65 ····· { FLOW 45° TO AXIS OF PIER b = PROJECTED WIDTH NALA SAND 1.3 m.m.



A-4

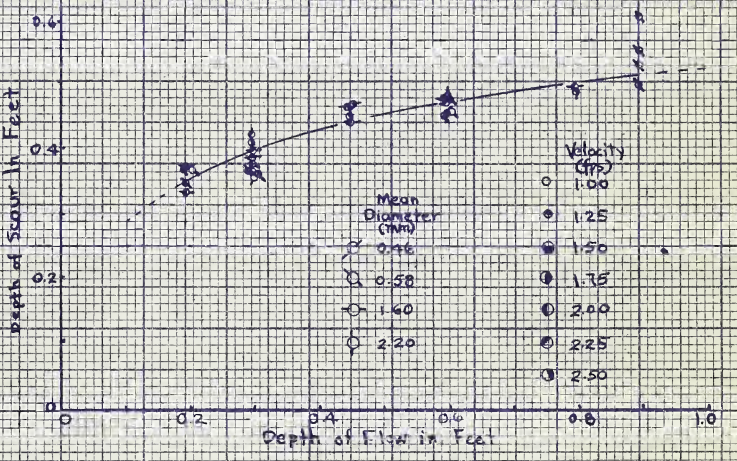


Fig. 10 - Effect of flow and sand characteristics

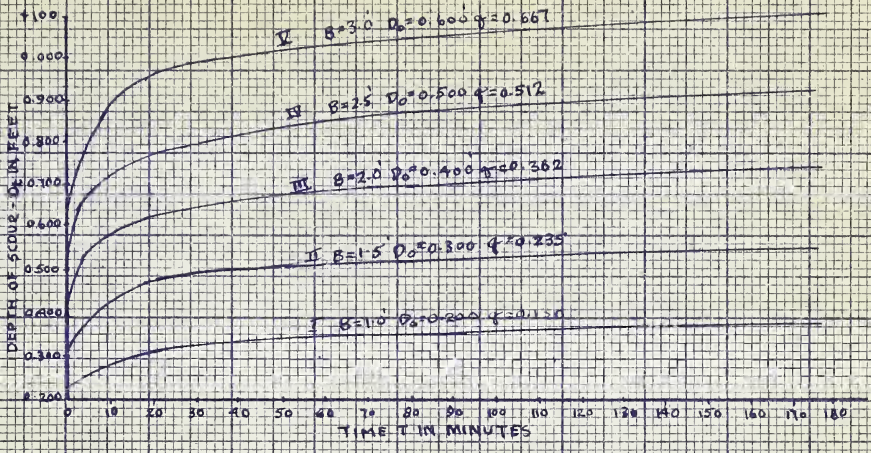

DEPTH OF SCOUR - D_s IN FEET
TIME T IN MINUTES
V B=3.0 D_o=0.600 q=0.667
IV B=2.5 D_o=0.500 q=0.512
III B=2.0 D_o=0.400 q=0.362
II B=1.5 D_o=0.300 q=0.235
I B=1.0 D_o=0.200 q=0.130

FIG 3 (a)

Curves between depth of scour and time for different q_s

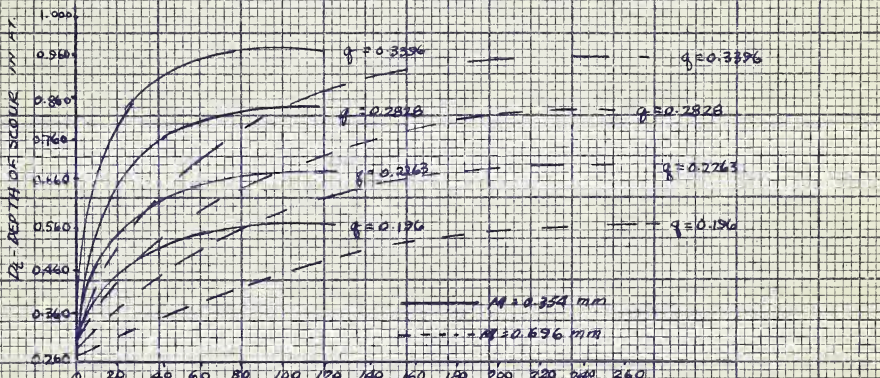


FIG. 4 (a)

Curves between D_t and time for different discharges

Part II – Layout
Photo. 1

Sounding grid

Photo 2

Initial - Run

Photo 3

Photo 4

Initial - Run 2

Photo 5

Photo 6

Initial-Rou

Photo 7

Final Run 3

Photo 8

Photo 9

Layout with spur

Photo 10

Photo 11

Photo 12

Photo 13

Photo 14

Photo 15

Photo 16

Initial - D

Photo 11

Final - Run 4

Photo 18

Final - Run 5

Photo 19

Photo 20

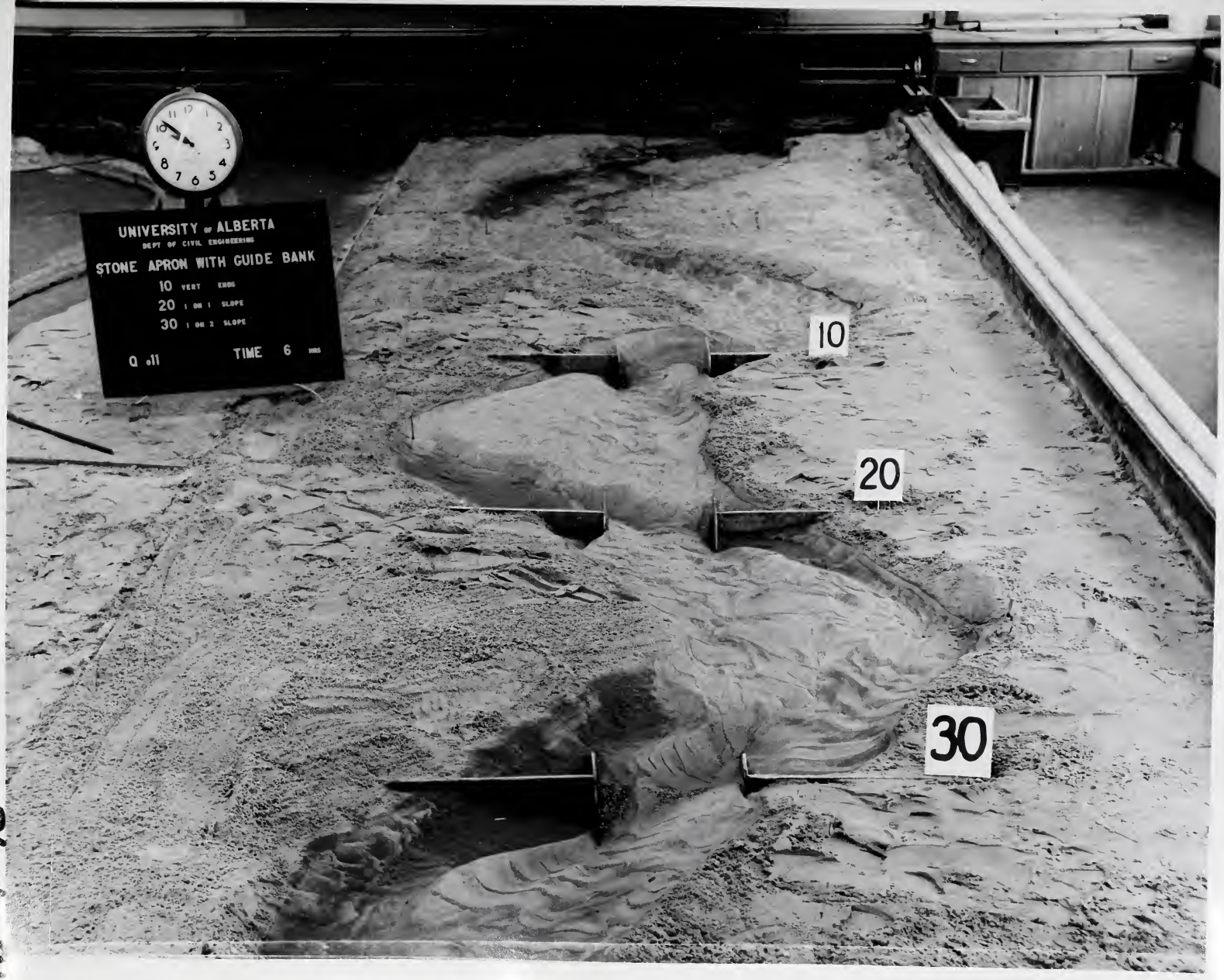

UNIVERSITY of ALBERTA
DEPT OF CIVIL ENGINEERING
STONE APRON WITH GUIDE BANK
10 VERT ENDS
20 1 ON 1 SLOPE
30 1 ON 2 SLOPE
Q =11
TIME 6 HRS
10
20
30

Photo 21

UNIVERSITY OF ALBERTA
DEPT OF CIVIL ENGINEERING
STONE APRON WITH GUIDE BANK
10 VERT ENDS
20 1 ON 1 SLOPE
30 1 ON 2 SLOPE
Q 0.11 TIME HRS
10
20
30
60 hrs

Photo 22

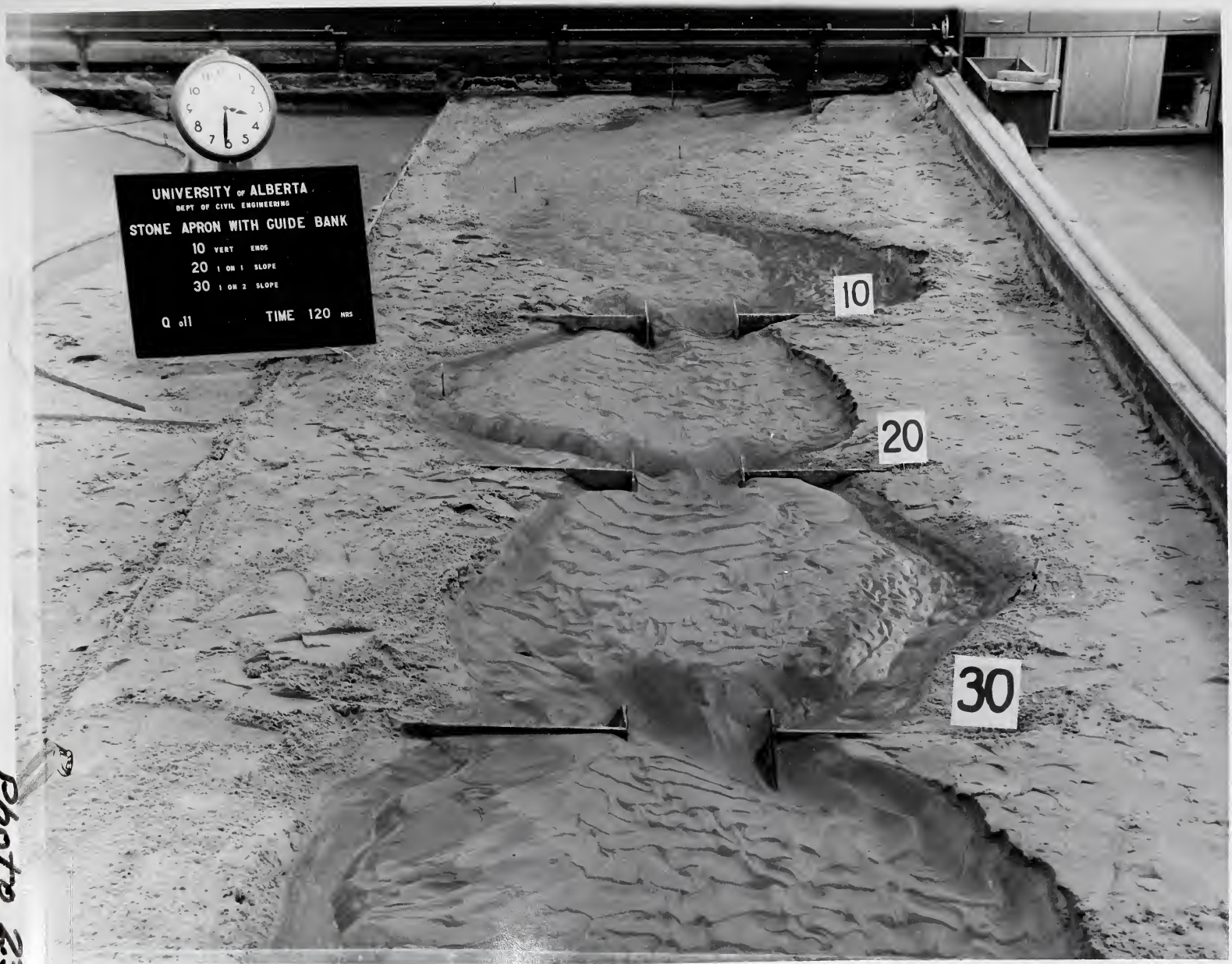

UNIVERSITY OF ALBERTA
DEPT OF CIVIL ENGINEERING
STONE APRON WITH GUIDE BANK
10 VERT EMOS
20 1 ON 1 SLOPE
30 1 ON 2 SLOPE
Q o11
TIME 120 HRS
10
20
30

Photo 23

UNIVERSITY OF ALBERTA
DEPT OF CIVIL ENGINEERING
STONE APRON WITH GUIDE BANK
10 VERT ENDS
20 1 ON 1 SLOPE
30 1 ON 2 SLOPE
Q .11
TIME 165 MIN
10
20
30

Photo 24

Photo 25

Photo 26

Photo 21

Photo 20

165 hrs.

Photo 29

Photo 30

Photo 31

Photo 32

120 hrs

Photo 33

165 hrs.

Photo 3

Photo 35

Photo 30

Photo 37

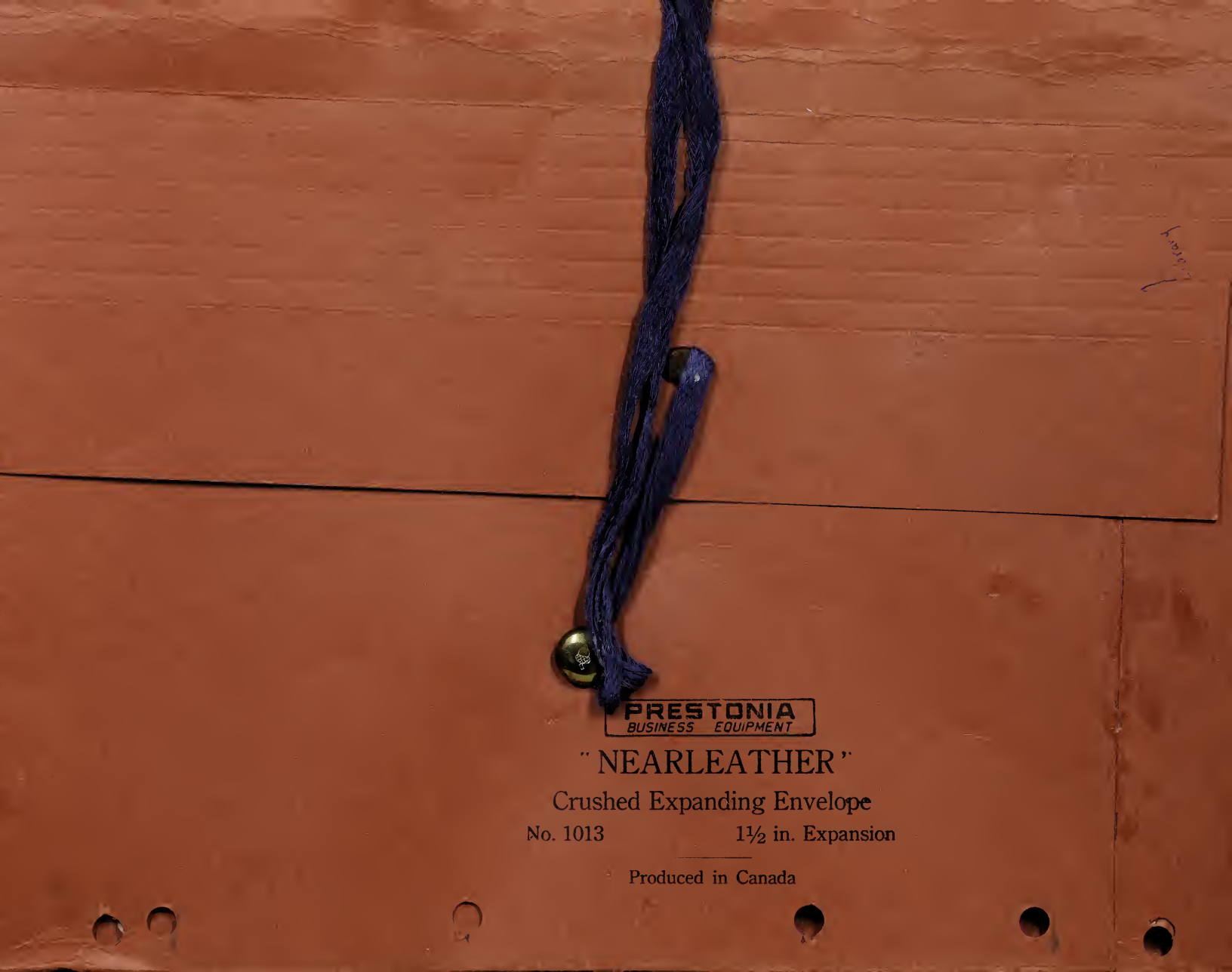
Photo 30

165 hrs.

Photo 39

Distorted Scour Hole

Photo 40



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